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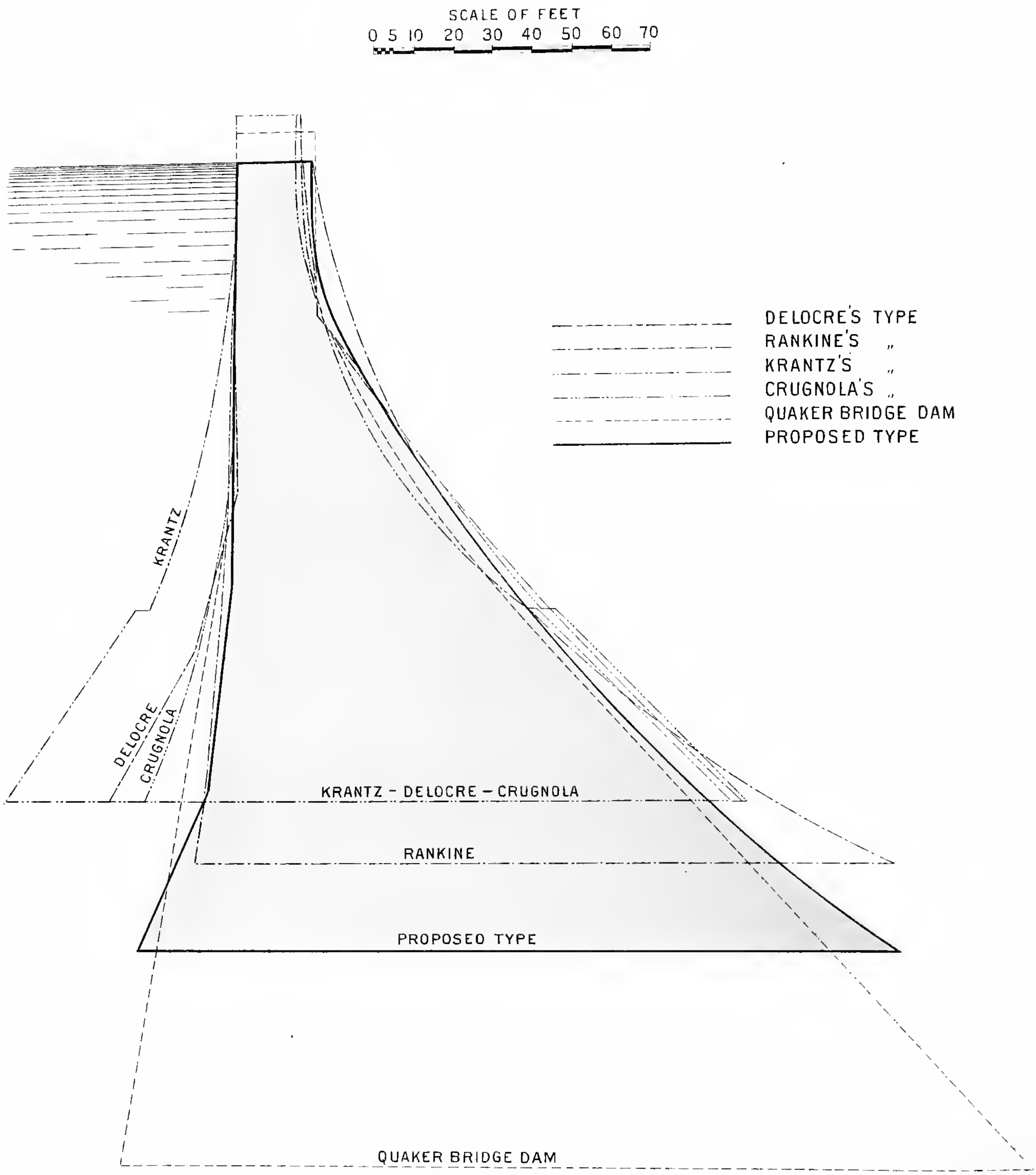
COMPARISON OF PROFILE TYPES



THE DESIGN

AND

CONSTRUCTION OF DAMS

INCLUDING

MASONRY, EARTH, ROCK-FILL, AND TIMBER STRUCTURES

ALSO

THE PRINCIPAL TYPES OF MOVABLE DAMS

BY

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PREFACE TO THE FIRST EDITION

THE great advantages to be derived from large storage reservoirs, built for regulating the flow of a river, for irrigation purposes, or for domestic water supply, have led within recent years to the construction of a large number of such works in various parts of the world. Where water having great depth is to be retained, it would be extremely hazardous to rely on earthen dams, as numerous failures of such works have been recorded, and walls of masonry are, therefore, employed.

The successful completion of the Furens Dam (164 feet high) in 1866 was soon followed by that of many similar structures in France, Algiers, and Italy. In the United States a concrete dam (170 feet high) is being built near San Francisco; the Sodom Dam (70 feet high) has been commenced on the East Branch of the Croton River; and the Quaker Bridge Dam, which will surpass all existing dams in height, has been designed to form an immense storage reservoir for the city of New York.

While the practical importance of the subject of masonry dams seems to be steadily growing, the engineer who may be entrusted with the design of such works will find the theoretical study of the best form of profile for a masonry dam very disheartening. How widely the types proposed by eminent engineers differ from each other is shown in our frontispiece.

The theory of masonry dams is based upon a few simple principles and conditions; the mathematics, however, to which they give rise, when applied to the design of an economical profile, are rather appalling. Thus, if we follow the methods of the French engineers Sazilly and Delocre, we have to solve lengthy equations, some of them of the sixth degree. Moreover, there is always an uncertainty which equation is to be used, and the only way of determining this is by trial. If we wish to employ the method of Prof. Rankine, but change the data assumed by him, we have to make trials with the subtangent of a logarithmic curve. In contradistinction to these scientific methods, we find prominent engineers recommending trial calculations as the best practical solution of the problem.

The writer, when detailed by the Chief Engineer of the New Croton Aqueduct to make calculations for the proposed Quaker Bridge Dam, the height of which is to be 270 feet, after studying the existing methods of designing profiles and finding them for various reasons inapplicable to the case in view, finally arrived at the equations given in this book. They are easy to solve, being, with the exception of one cubic equation, of the first or second degree. The theoretical section of the Quaker Bridge Dam was calculated by these equations. As the construction of this gigantic dam,

which is likely to be commenced soon, may lead many persons to inquire how its profile was determined, the writer has thought that a book giving the details of the method employed, and information about masonry dams in general, might be of interest and practical value to engineers. It is with this view that the present work has been undertaken.

The text has been illustrated by numerous Plates and Tables, showing the form and strength of the various profiles discussed. Data of forty-four existing masonry dams have been collected in Table XXIII.

The investigations given in Chapter IV., relating to the effect of the weight of masonry upon the form of profile and the calculations for inclined joints, were suggested in connection with the proposed Quaker Bridge Dam by Mr. B. S. Church, Chief Engineer, and Mr. A. Fteley, Consulting Engineer.

In the preparation of this book the writer has been assisted by some of the engineers of the New Croton Aqueduct, who have become interested in these studies, and he wishes to express herewith his thanks to Mr. H. C. Alden and Mr. M. A. Viele, who have helped him to calculate the Tables, and to Mr. G. Bonanno and Mr. I. A. Shaler, who have rendered valuable aid in making the drawings and in collecting information about existing dams.

E. W., JR.

NEW YORK, April, 1888.

PREFACE TO THE THIRD EDITION.

SINCE the first edition of this work was published, the project of constructing a dam across the Croton Valley near the Quaker Bridge has been abandoned, another site, about $1\frac{1}{8}$ miles further up-stream, having been selected for the proposed structure. As the profile adopted for this reservoir wall is, however, practically the same as the one designed for the proposed Quaker Bridge Dam, the studies made for the latter, which are given in this book, may still be of interest.

The only change which has been made, therefore, in the present edition of this work has been to add a new Chapter and additional Plates, in order to bring the descriptions given of dams constructed up to date.

E. W., JR.

NEW YORK, Sept. 1, 1893.

PREFACE TO THE FOURTH EDITION.

THE first Part of this work was published in 1888 as a treatise on "The Design and Construction of Masonry Dams." It contains the results of the studies made by the author while engaged in making calculations for the design of the proposed Quaker Bridge Dam, and information about high masonry dams built in various countries. The book passed through three editions, the only changes made being the addition of an extract from the report of the experts who investigated the designs for the Quaker Bridge Dam, and of descriptions of dams recently constructed, in order to bring the information on this subject up to date. In the present edition the work has been enlarged so as to include the whole subject of dams, viz., masonry, earth, rock-fill, and timber structures, and, also, the principal types of movable dams.

The author has endeavored to add to the practical interest of the original book, which forms Part I of the present work, by inserting half-tones in the text, giving detailed descriptions and drawings of some of the high masonry dams recently built to form storage reservoirs for the water-supply of the city of New York, and by placing in the Appendix the specifications for the New Croton Dam, which will be by far the highest structure of its kind. To illustrate fully the manner in which the profile of this dam was calculated by the method explained in Part I, Chapter III, a practical example of the application of the equations used for this purpose has been given in the Appendix.

The subjects of earth, rock-fill, and timber dams are discussed in Part II. The principal types of movable dams are described in Part III. Since 1839 internal navigation in France has been much improved by means of structures of this kind. Thus far only a few movable dams have been built in the United States. As this subject has been attracting much attention in America of late, the author has thought it advisable to include it in the present edition.

The information given in this volume has been gathered from many sources. The authorities consulted are given in the Appendix (p. 239). The sources from which the illustrations in the book were obtained are generally mentioned in the text. The half-tones of the Furens and Vyrnwy dams were reproduced from plates contained in a report on high masonry dams, by Mr. George W. Rafter, which was included in the Annual Report for 1897 of the Engineer and Surveyor of the State of New York. In Plate B, photographs of the Gileppe Dam, which were kindly loaned the author by Mr. Alphonse Fteley, the Chief Engineer of the Aqueduct Commission of New York, are reproduced. Figs. 39 and 54 were taken from Mahan's "Civil Engineering." Figs. 19 to 21 and 23 were obtained from the 18th Annual Report of the U. S. Geological Survey. The author wishes to acknowledge here his obligation to Mr. F. H. Newell, the hydrographer of this survey, who kindly furnished him with electrotypes of all the plates and figures contained in the reports of the U. S. Geological Sur-

vey which have been reproduced in this book. He wishes, also, to express his thanks to Mr. Edward A. Bond, State Engineer of New York, who enabled him to obtain electrotypes of Plates A and C.

The author hopes that the new matter added to the book will increase its usefulness and that it may be of practical help to members of the profession who cannot avail themselves of the information contained in large technical libraries.

E. W.

NEW YORK, July 1, 1899.

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DESIGN AND CONSTRUCTION OF MASONRY DAMS.

CHAPTER I.

INTRODUCTION.

THE remains of ancient works still existing in India and Ceylon bear evidence that the construction of reservoirs for storing water dates from a very remote period of history. The ordinary manner of forming these basins, some of which were of vast extent, consisted in closing a valley by dams of earth; and it was not until comparatively recent times that walls of masonry were employed for such purposes. This method of construction seems to have been first adopted in the southern part of Spain, where, about the sixteenth century, large reservoirs for irrigation were constructed. Much as these early masonry dams excite our admiration by their great dimensions and massiveness, their proportions demonstrate that their designers had no correct conception of the forces to be resisted. By a faulty distribution the great mass of material in some of these walls produces undue strains in the masonry or on the foundation, becoming thus a source of weakness rather than of strength. Prior to the middle of the present century most masonry dams were built according to defective plans. It has been shown, indeed, that some of these walls would have been stronger had their positions been reversed, the down-stream face being turned up-stream.

The French engineers advanced the first rational theory of masonry dams, and proved its correctness by applying it in the construction of some of the highest and boldest reservoir walls of the present time. By means of the great storage basins thus obtained, they control the flow of rivers, retaining the excess of water during the period of flood for the time of drought. The havoc due to inundations may thus be largely prevented, and replaced by all the benefits resulting from irrigation, domestic water supply, and the furnishing of a cheap motive power.

De Sazilly.

Before entering upon any mathematical details, we will glance briefly at the different steps that have been made in the development of the theory of masonry dams. The first writer who investigated this subject in a satisfactory manner was M. de Sazilly. His memoir on the design of reservoir walls appeared in the "Annales des Ponts et Chaussées" for 1853. According to this writer the safety of a masonry dam depends upon the compliance with the following two conditions:

1st. The pressures sustained by the masonry or its foundation must never exceed a certain safe limit.

2d. There must be no possibility of any portion of the masonry sliding on that below, or of the whole wall moving on the foundation.

To devise a formula containing both of the above conditions of safety would be difficult, if not, indeed, impossible. However, M. de Sazilly states that no masonry dam has been known to fail by sliding, and he therefore recommends that the profile of a dam be designed solely with reference to the first of the conditions, leaving it to a subsequent trial to determine whether the second has been fulfilled. This will generally be found to be the case, especially if the assumed limit of pressure is not very high and the dam has a considerable top-width. Should we find that the wall or part of it might slide, then Sazilly's method would be to increase the thickness of the profile by recalculating it for a lower limit of pressure. This writer pointed out that, in determining the maxima pressures in the masonry or on its foundation, two extreme cases must be considered:

1st. When the reservoir is full.

2d. When the reservoir is empty.

These two conditions give the extreme positions of the lines of pressure* in a dam, the first causing the maximum pressure in any horizontal plane to be at the front (down-stream) face of the wall, and the second producing them at the back (up-stream) face. The practical considerations of economy require that a dam should contain the minimum amount of material consistent with safety. Having established a fixed limit of pressure, Sazilly's ideal profile is that in which the maxima pressures in both faces just reach the limit. He called this "the profile of equal resistance." In attempting to find formulæ for determining its form, Sazilly experienced no difficulty in obtaining the correct differential equations, but found it impossible to integrate them, and had therefore to abandon [illegible] finding the proper curves for the faces of the "profile of equal resistance." The difficulties of the integration may, however, be avoided by substituting for curved outlines polygonal or stepped faces.

In either case we must assume the dam, for the purposes of calculation, to be divided into courses by horizontal planes. The smaller the depth of these courses, the closer the "profile of equal resistance" will be approached, the approximate types involving always a slight excess of masonry. Against polygonal faces Sazilly urges the following objections:

1st. The angles form points of weakness.

2d. The gentle slopes of the faces favor vegetation of parasitical plants, which injure the masonry.

3d. Such a wall would have a bad appearance and would be difficult to execute.

For the above reasons he recommends as the best practical type a stepped profile, such as shown in Plate I and Table I, for which he gives formulæ.

Delocre.

The next engineer who advanced a method for determining the profile of a masonry dam was M. Delocre. The frequent inundations in the valley of the Loire led the French engineers to plan large reservoirs for retaining the flood-water. Good locations for such works were readily to be found in the upper valleys of the branches of this river, but sufficient storage capacity could only be obtained by constructing dams up to 50 metres (164 ft.) in height. To have formed them of earth would

* The line of pressure (called also line of resistance) is a line intersecting each joint of a structure (whether it be real or imaginary) at the point of application of the resultant of all the forces acting on that joint.

have been extremely hazardous, and walls of masonry were therefore decided upon. M. de Græff, the Chief Engineer of the "Département" in which these reservoirs were to be located, assigned the study of the best type of profile to M. Delocre, and it is upon this engineer's investigations and formulæ that most of the high dams built within recent times have been based. Starting with the same conditions and fundamental formulæ as Sazilly, Delocre arrived at different conclusions. He demonstrated that a stepped profile involved considerable waste of material and required, moreover, an expensive class of masonry for the steps. He argued that the objections raised by Sazilly against polygonal faces would lose their force if only a few changes of slope were employed, and that by adopting such outlines a considerable economy might be effected.

Plate II. and Table II. give the type of profile recommended by Delocre. For sake of comparison he made calculations for two profiles of a dam 50 metres high, one according to Sazilly's method, and the other according to his own, basing them upon a weight of masonry of 125 lbs. per cubic foot, and on a limiting pressure of 6 kilos. per square centimetre (6.15 tons of 2000 lbs. per sq. foot). In Sazilly's type this pressure is only reached at the re-entrant angles of the steps; in Delocre's, only at the vertices of the angles in the faces. The calculations are made for one lineal metre of wall, which is supposed to resist the thrust of the water simply by its weight. Tables I. and II. show that these profiles differ very little from each other as regards stability or resistance to sliding, but the following figures prove that an economy results from adopting Delocre's type:

	Area of profile, in square metres.	Exposed surface for one lineal metre of wall, in square metres.
Sazilly's type, ...	1028.75	152.15
Delocre's type, ...	995.30	119.70
	33.45	32.45

Delocre has given lengthy formulæ for calculating profiles according to his method, and also investigated the additional strength which might be obtained by building a dam on a horizontal curve in plan. The results of his studies were known in 1858, and formed the basis of the design for the Furens dam near St. Etienne, a reservoir wall 50 metres in height. It was not, however, until after the completion of this work that M. Delocre published in the "Annales des Ponts et Chaussées" for 1866 a memoir giving the details of his researches.

Rankine.

To trace the history of our subject chronologically, we must now turn to an English writer for the next marked advance. In connection with some proposed reservoirs for the city of Bombay, the question arose of deciding between the respective merits of earthen and masonry dams. In order to obtain the opinion of a high scientific authority on this subject the question was submitted to Prof. W. J. M. Rankine, who was also requested to make a rigid mathematical investigation of the best form of profile for a masonry dam. The report* written by Prof. Rankine in response to this request is very complete. His views of the considerations that ought to determine the design for such a dam will readily be accepted. While Rankine recommends that the profile should be determined mainly by the principles

*See Prof. Rankine's "Miscellaneous Scientific Papers," 1881.

laid down by the French engineers, he improves their methods in some respects. Thus these engineers, in calculating the maxima pressures in the masonry, had only considered the vertical component of the resultant pressure of the forces acting at any joint. They therefore assumed the same limit for the intensity of vertical pressure at both faces of the wall. Prof. Rankine says, however: "It appears to me that there are the following reasons for adopting a lower limit at the outer than at the inner face. The direction in which the pressure is exerted amongst the particles close to either face of the masonry is necessarily that of a tangent to that face; and, unless the face is vertical, the pressure found by means of the ordinary formulæ is not the whole pressure, but only its vertical component; and the whole pressure exceeds the vertical pressure in a ratio which becomes the greater, the greater the 'batter,' or deviation of the face from the vertical. The outer face of the wall has a much greater batter than the inner face; therefore, in order that the masonry of the outer face may not be more severely strained when the reservoir is full than that of the inner face when the reservoir is empty, a lower limit must be taken for the intensity of the vertical pressure at the outer face than at the inner face. . . ."

This eminent writer did not attempt to determine the ratio which the limits of the vertical pressure at the front and back face ought to bear to each other, by any mathematical deduction, as he deemed the data upon which it would have to be based too uncertain. In choosing the limits of pressure for the profile accompanying his report (see Plate III. and Table III.) he was guided entirely by what experience had proved to be safe, and adopted:

	Limit of vertical pressure, in pounds per square foot.
For front (down-stream) face, ...	15,625
For back (up-stream) face, ...	20,000

The same reasoning which led Rankine to recommend a lower limit of vertical pressure for the front face than for the back face induced him to make the pressures at the front face diminish as the batter increases. Here, too, he followed practical examples, as he thought it impossible in our present state of knowledge to deduce a law for this diminution. He designed his profile therefore in such manner that the maximum pressure at a depth of 150 feet would equal the pressure at the same depth in the Furens dam; viz., 6½ kilos. per square centimetre, or about 6.65 tons of 2000 lbs. per square foot. Below this depth the maxima pressures diminish gradually.

Another principle pointed out by Prof. Rankine is that no tension must be allowed in the masonry. Theoretically this would occur (as will be shown in Chapter II.) whenever the line of pressure lies at any point outside of the centre third of the profile. The stability of the dam against overturning depends upon the position of the line of pressure. For the above reasons Rankine limits these lines (reservoir full or empty) to the centre third of the profile.

The conditions given by Rankine do not prescribe any definite form of profile, but when we add the consideration of economy, requiring the minimum amount of material consistent with safety, the choice of form becomes very limited. The types of Sazilly and Delocre involve very lengthy calculations. Prof. Rankine endeavored to find simpler

formulæ. One of the effects of his using a higher limit of pressure for the back face than for the front is to reduce the batter of the former considerably from that of the French types. The vertical component of the water-pressure on the back of dam can therefore add but little to the stability of the wall. Prof. Rankine neglected this component in his formulæ, as the slight error thus introduced would be in the direction of safety, and also simplifies largely the mathematical investigation.

As regards the profile (Plate III.) accompanying his report, he says: "In choosing a form in order to fulfil the conditions without any practical important excess in the expenditure of material beyond what is necessary, I have been guided by the consideration that a form whose dimensions, sectional area, and centre of gravity under different circumstances are found by short and simple calculations, is to be preferred to one of a more complex kind when their merits in other respects are equal, and I have chosen logarithmic curves for both the inner and outer face, the common subtangent being 80 feet for both."

The formulæ given by Prof. Rankine for determining the thickness, area, etc., of this profile are certainly extremely simple; but then they produce only this one profile, whose dimensions might as well be calculated once for all. Change the data upon which it is based, such as the weight of masonry, limiting pressures, etc., and the simplicity of this method disappears. Rankine states that his general formulæ which we would have to employ in such a case are "incapable of solution by any direct process." They can, however, be solved approximately by a process of trial and error, involving the higher mathematics.

If by means of logarithmic curves a profile differing but little from the exact theoretical type could be obtained, there would be no objections to the use of such approximate methods. Sazilly had demonstrated that a wall sustaining only its own weight would contain the minimum amount of material, consistent with a fixed limit of pressure, by having symmetrical faces, which would be vertical until the limit of pressure were reached, and would then follow logarithmic curves. But similar outlines will not give the best profile for a dam resisting water-pressure, as will be shown in Chapter IV.

Prof. Rankine seems to have been the only English writer who investigated the theory of masonry dams mathematically. Since he wrote his report some additional memoirs on this subject have appeared in the "Annales des Ponts et Chaussées." Thus M. Bouvier, in writing a description of the Ternay dam,* reached Rankine's conclusions that the pressures near the front face of the wall would be increased by their inclined direction, although he based this fact, not on the batter of the face, but upon the direction of the resultant pressure. He introduced modifications in the formulæ used by Sazilly and Delocre for calculating the maxima pressures, which we will explain in Chapter II.

Another analytical method of calculating the profile of a masonry dam was advanced by M. Pelletreau in the "Annales des Ponts et Chaussées" for 1876, 1877, and 1879. This writer adopted the same basis as Sazilly and Delocre, placing no limits to the positions of the lines of pressure. By an intricate investigation involving the higher mathematics he found a simple series which expresses the thickness of a dam

* Annales des Ponts et Chaussées, Aug. 1875.

at any depth, so long as the back face remains vertical; but for the case when both faces must be battered he did not succeed in finding a general formula.

In addition to the analytical methods already mentioned, M. de Beauve has given in his "Manuel de l'Ingénieur des Ponts et Chaussées" a graphic solution, which leads to accurate results but is very laborious.

The methods of determining profiles already described are all which we have been able to find, excepting some purely empirical formulæ by Molesworth and others, and a method by Mr. W. B. Coventry, consisting partly in the use of equations and partly in trial-calculations. This engineer states in his memoir on "The Design and Stability of Masonry Dams:"* "Owing to the indeterminate nature of the problem, it seems impossible to construct a general formula for calculating the dimensions of a dam, and the method usually followed consists in assuming an approximate profile, and then testing its stability by a graphic resolution of forces. If found defective the profile is altered, and the graphic process repeated until a sufficiently exact result is obtained."

The stability of a masonry dam depends upon a few simple principles; the mathematics, however, to which they give rise, when applied to the design of a profile, are exceedingly complicated and lengthy. How unsatisfactory the present formulæ devised for that purpose are, may be judged from the fact that Prof. A. R. Harlacher of Prague, in a report on a proposed dam near Komotau (Bohemia), written in 1875, recommends trial-calculations as the best practical method of finding the correct profile for a masonry dam. In this manner he designed the profile given in Plate VI. and Table VI. Another German engineer, F. Kuhn, advances the same opinion.†

M. Krantz ‡ and M. Crugnola § have published profile-types for dams of various heights, which were probably found by trial. Neither of these engineers gives any formulæ for this purpose. The types they proposed for a dam 50 metres (164 feet) high are shown respectively in Plate V. and Table V., and in Plate VII. and Table VII.

While a correct profile for a masonry dam may doubtless be found by a sufficient number of trials, such a method is very laborious and unsatisfactory. Impossible as it may be to determine at once the proper outlines for a profile which shall contain the minimum area consistent with the given conditions, there are no great mathematical difficulties involved in calculating its thickness at intervals, commencing at the top. To obtain the minimum area these intervals should be infinitely small. For practical purposes we can find a profile which shall approach the true theoretical type as closely as may be desired, by making the intervals at which the calculations are made sufficiently small.

The profile resulting from this method will have polygonal outlines, involving many changes of slope. As it fulfils all the given conditions and contains, at the same time, practically the minimum area consistent therewith, we shall call it *the Theoretical Profile*. It might serve as a design for a dam, were it not for constructive objections to the many changes of batter in the faces. To obtain a profile which can be readily

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* Paper 2110, vol. 85, Proc. Inst. C. E. (session 1885–86).

† F. Kuhn, "Die Thalsperre der Gileppe bei Verviers," published in *Der Civil Ingenieur* for 1879.

‡ Étude sur les murs de réservoir. Paris, 1870.

§ Muri di Sostegno e Traverse dei Serbatoi d'Acqua. Torino, 1882.

executed and also offers a pleasing effect, we have only to fit a few simple curves or straight lines to the theoretical form, reaching thus *a Practical Profile.* Small changes made for this purpose will have but a trifling effect upon the stability of the dam; and while the practical profile may not satisfy the given conditions rigidly, it will certainly do so practically.

The methods proposed by the eminent engineers already mentioned give but approximations to the correct theoretical form. Closer results may be obtained by following the method we have just explained. The equations we shall give for this purpose are exceedingly simple, being all quadratic with the exception of one of the third degree, which will seldom be used. The profile for the proposed Quaker Bridge Dam (see Plate LXXIII.) was determined in this manner.

Before explaining our method in full, we will first examine in the next chapter the fundamental formulæ used by all writers for determining the distribution of stress in a wall of masonry.

CHAPTER II.

DISTRIBUTION OF PRESSURE IN A WALL OF MASONRY.

SCIENCE has not yet revealed the laws which the internal stresses in a mass of masonry follow. We are therefore obliged, in treating of masonry dams, to resort to some safe hypothesis which shall furnish results approximately correct. All mathematical formulæ for masonry dams have thus far been established by considering these walls as rigid and composed of homogeneous masonry. This hypothesis involves two inaccuracies, as masonry is always more or less elastic, and seldom perfectly homogeneous. We shall show in this chapter that by assuming a dam to be rigid we exaggerate, in all probability, the pressures in the weakest part of the wall, namely, near the faces,—and make thus an error in the direction of safety. By careful inspection during construction we may obtain masonry which shall be practically homogeneous. The above hypothesis may therefore be safely accepted, and we shall in future consider masonry dams as forming rigid homogeneous monoliths.

Another assumption which is generally made is that a dam will resist the thrust of the water simply by its own weight. It follows from this that in studying the conditions of equilibrium of such a wall (every part of which is supposed to be built according to the same profile) we need only consider a section one foot long.

For the present investigations we will assume a dam built according to some ordinary type, having sloping faces and a rectangular base, resting on a horizontal foundation. Whether the reservoir be empty, partially or totally filled, the given section of wall will be acted upon by symmetrical forces. Their resultant must lie in a vertical plane, perpendicular to the faces of the wall, and bisecting the given section. When the reservoir is empty the resultant will be the total weight of the wall acting vertically through its centre of gravity. We will confine ourselves first to this case and investigate the laws of distribution of the pressure on the base of the wall. Let us suppose the dam to be rigid and built upon a perfectly elastic foundation.

When the resultant pressure passes through the centre of gravity of the base (which in the given case is also its geometrical centre) the pressure will be uniformly distributed over the foundation, by compressing which the dam will settle evenly, its base remaining horizontal. When, however, the resultant pressure does not pass through the centre of gravity of the base, it will no longer be uniformly distributed on the foundation. The eccentric resultant by throwing more pressure on one edge of the base than on the other will cause unequal settling and therefore tilt the dam. Owing to the rigidity of the wall and the elasticity of the foundation, the pressures on the latter will now follow the laws of a uniformly varying stress, and may be represented by the ordinates between a horizontal and an inclined line.

The reaction of the foundation from face to face of wall may be shown graphically by a plane figure, whose area will represent the total pressure, whose centre of gravity will lie

in the line of action of the resultant pressure, and whose vertical ordinate at any point will give the corresponding intensity of stress. A uniformly distributed pressure would be represented graphically by a rectangle as in Fig. 1. A uniformly varying pressure will be shown graphically by a trapezoid or triangle, depending upon the position of the result-

Fig. 1

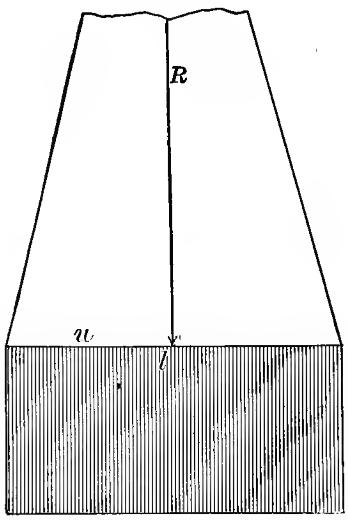
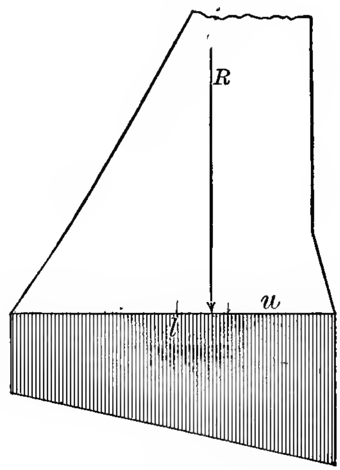


Fig. 2



ant pressure, which must pass through the centre of gravity of the figure, viz.: When the eccentricity of the resultant is less than one sixth the width of the base, the reaction of the foundation will be shown by a trapezoid, Fig. 2. Should this eccentricity just equal one sixth the width of the base, the trapezoid would become a triangle, Fig. 3. When the resultant is still nearer to one edge, the dam will be tilted so much that part of the base

Fig. 3

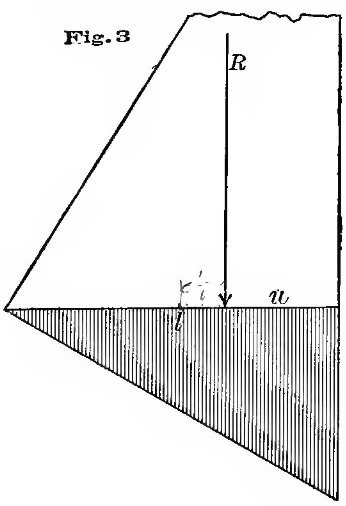
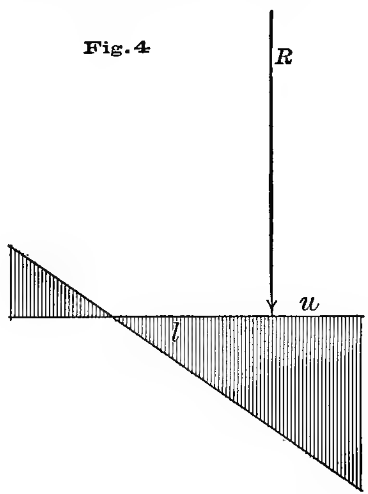


Fig. 4



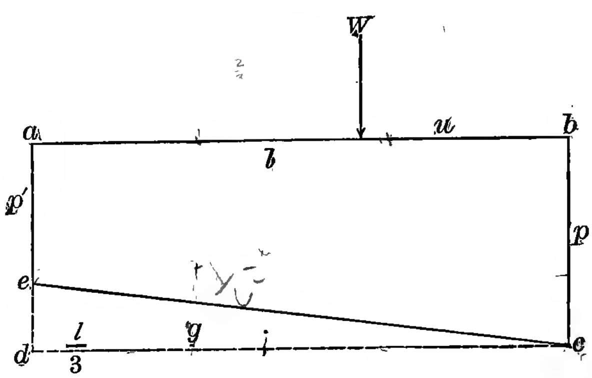
will have to bear the whole pressure, the remaining portion being raised entirely from the ground. The reaction in this case will also be represented by a triangle as in Fig. 4. Should the adhesion between the foundation and the base prevent the latter from being partially raised from the ground, then part of the base will have to sustain tension. This will modify the distribution of the pressure, which, however, will still form a uni-

formly varying stress. To illustrate the above laws, imagine a rigid plank floating in water and bearing a movable load. As this latter is shifted from the centre towards either end, each of the above cases will arise, the water-pressure under the plank representing the reaction of the foundation.

We have thus far considered the foundation as elastic, but will now suppose it to be rigid. It can no longer be compressed by the weight of the dam; but as the tendency to cause compression remains the same whether the foundation be rigid or elastic, it is rational to assume the same distribution of pressure for both cases.

The laws of distribution of pressure, given above, were first indicated by M. Méry in his memoir on the stability of arches, and were perfected by M. Bélanger in the course of Applied Mechanics taught by him at the École des Ponts et Chaussées.* The formulæ resulting from these laws may be derived in many different ways. The following solution will be found to give the usual formulæ by a short method. Both the dam and the foundation are assumed as rigid.

Fig. 5



Let $W =$ the total pressure on the base ab;

$u =$ the distance of W from the nearer edge b;

$p =$ the maximum intensity of pressure on the foundation;

$p' =$ the minimum intensity of pressure on the foundation;

$l =$ the width of the base ab;

$g =$ the centre of gravity of triangle ced.

First let $u > \frac{l}{3}$.

$$\text{Trapezoid } abce = \text{the reaction of the foundation}$$
$$= abcd - ecd$$
$$= pl - \frac{l}{2}(p - p').$$

As W and the reaction of the foundation are in equilibrium, the algebraic sum of their moments about any point must be zero.

Taking moments about the point g, we find, since the moment of the triangle cde is o,

$$\frac{l^2p}{6} - W\left(\frac{2}{3}l - u\right) = 0;$$

whence
$$p = \frac{2W}{l}\left(2 - \frac{3u}{l}\right). \quad \text{(A)}$$

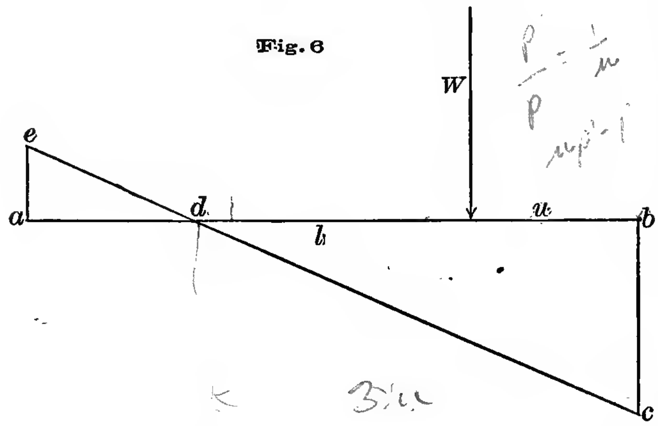
When $u = \frac{l}{3}$, the trapezoid of reaction becomes a triangle, and we have

$$p = \frac{2W}{l}. \quad \text{(B)}$$

* M. Sazilly's memoir on reservoir walls in the "Annales des Ponts et Chaussées" for 1853.

When $u < \frac{l}{3}$ we should have, according to the laws of a uniformly varying stress, a positive and a negative triangle, the former representing the pressure on the foundation, the latter the tension on the base. As it would, however, be unsafe to depend upon the tension in the masonry, it is best to neglect it in calculating the pressure on the foundation. Fig. 6 shows this case.

Fig. 6



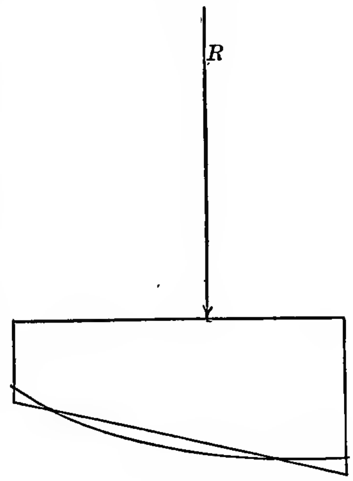
Neglecting the tension ade, and taking moments about b, we obtain

$$Wu - \frac{3pu^2}{2} = 0;$$

hence $$p = \frac{2W}{3u}. \quad . \quad . \quad . \quad . \quad \text{(C)}$$

Formulæ A, B and C are correct for a rigid dam resting upon a rigid or elastic foundation. But now let us suppose the masonry itself to be elastic. We have no exact knowledge of the manner in which pressures would be distributed in such a body. However, in the practical cases with which we have to deal we can indicate how the formulæ A, B and C would have to be modified. Unless the height of a dam be very insignificant, one or both faces will be sloped or stepped. The compression of the masonry at the base of a dam, resulting from its own weight, will depend mainly upon the column of masonry directly over any given part. The central portions of the base will, therefore, be compressed more than those near the edges, and consequently the diagrams of the reaction of the foundation will be modified somewhat as shown in Fig. 7 by the curved line. The pressures near the edges of the base will be less and in the other parts greater than those calculated for a rigid dam. We certainly cannot conceive of the opposite to this taking place. Now, the central portions of the masonry, owing to the lateral support they receive, can bear safely much greater pressures than the masonry at the faces. Thus, we conclude that the inaccuracies involved in using formulæ A, B and C will be on the safe side. These formulæ may evidently be applied in finding the pressures in the masonry at any given plane; for we may consider the part of the wall above the plane as forming the dam proper, and the lower part as the foundation.

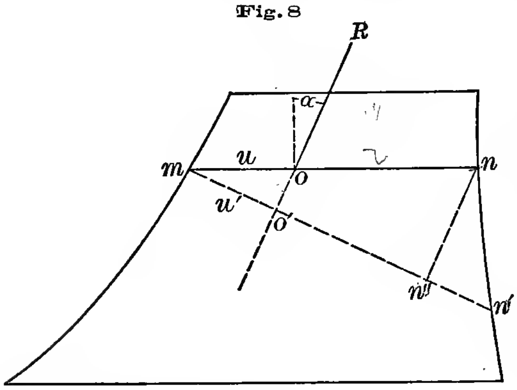
Fig. 7



Thus far we have only considered the distribution of a vertical resultant on a horizontal plane. When, however, the resultant is inclined (as will always be the case when the reservoir is filled) it can be resolved into two components, one parallel with and the other normal to the given plane. The former will be opposed by the resistance of the dam to sliding or shearing. The latter will be distributed in accordance with formulæ A, B or C. This method of calculating the pressures in the masonry at any horizontal joint has

been adopted by most of the early writers on the subject of dams. But, although the results obtained are correct as regards the distribution of the normal pressure, this method does not determine the maxima pressures to which the masonry is subjected. As already stated in Chapter I., Prof. Rankine argues that the pressures near the faces of the wall will be tangential to them, and that therefore, in considering the pressures normal to a given horizontal joint, we have only taken part and not the whole pressure.

Fig. 8



As he thought it impossible to find the real maxima pressures by any theoretical means, he advised the use of the old method of resolving the resultant pressure into a normal and parallel component; but adopted different limits of pressure for the front and back face of the wall, on account of the difference in their batters.

The French engineer M. Bouvier claimed that the maxima pressures would depend upon the inclination of the resultant, and therefore modified the formulæ already given as follows:*

Let R = the resultant pressure on an imaginary joint mn;
α = the angle of inclination of R from a vertical line;
l = length of joint mn;
l' = length of joint mn', which is perpendicular to the direction of R;
$u = mo$;
$u' = mo'$.

M. Bouvier considers the pressure of R to be distributed in the following manner on the joint mn' to which it is normal: He neglects the weight of triangle mnn'', and considers it simply as transmitting the pressure of R to mn''. He also assumes that no pressure will pass through the triangle $nn'n''$. The whole pressure of R will, therefore, be distributed on mn'' in accordance with the laws embodied in formulæ A, B and C. We must, however, substitute

for l, ... $l \cos \alpha = mn''$;
for u, ... $u \cos \alpha$;
for W, ... R.

The resulting formulæ will be:

$$p' = 2\left(2 - \frac{3u}{l}\right)\frac{R}{l \cos \alpha}; \quad \ldots \ldots \quad \text{A}'$$

$$p' = \frac{2R}{l \cos \alpha}; \quad \ldots \ldots \quad \text{B}'$$

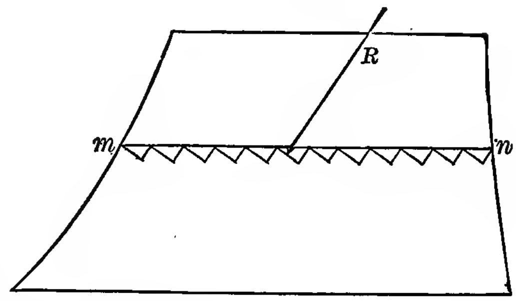
$$p' = \frac{2R}{3u \cos \alpha}. \quad \ldots \ldots \quad \text{C}'$$

The pressures obtained by formulæ A′, B′ and C′ are evidently always in excess of those derived by A, B and C. M. Bouvier states that M. Blanc arrived at the same

* 'Calculs de résistance des grands barrages en maçonnerie." Annales des Ponts et Chausseés, Aug. 1875.

conclusions from a more general discussion of the subject (see memoir No. 242 in the "Annales des Ponts et Chaussées" for 1869). The writer is also informed that M. Guillemain in his lectures at the École des Ponts et Chaussées derives A′, B′ and C′ by considering only the projections normal to R of the portions of the joint mn as shown in Fig. 9.

Fig. 9



All mathematical methods of designing dams are based upon a distribution of pressure according to the laws explained in this chapter. Great caution must be used, however, in applying the formulæ given to cases where the resultant pressure is sufficiently eccentric to produce tension. In a well-designed dam this condition ought never to exist. Where part of the masonry is under pressure and part in tension, there will be great uncertainty in determining the distribution of the stresses. The usual method in such cases has been to neglect the tension and to use formula C for finding the pressure.

In the method of designing profiles which will be given in the next chapter, we shall limit the position of the lines of pressure to the centre third of the profile, avoiding thus all tension and calculating the maxima pressures by formula A or B.

CHAPTER III.

THEORETICAL PROFILES.

IN the following investigations—

"Front" will signify "down-stream."

"Back" will signify "up-stream."

P will denote the line of pressure, reservoir full.

P' will denote the line of pressure, reservoir empty.

The unit of weight will be one cubic foot of masonry.

All linear dimensions will be expressed in feet.

a = the top width of the dam.
x = the unknown length of a joint of masonry.
l = the known length of the joint above x.
h = the depth of a course of masonry (assumed generally = 10 feet).
u = the distance of P from the front edge of the joint x. u = dist. of
n = the distance of P' from the back edge of the joint x. n = dist.
m = the distance of P' from the back edge of the joint l.
v = the distance between P and P' at the joint x.
f = the coefficient of friction for masonry on masonry.
c = the cohesion of the masonry per square unit.
r = the specific gravity of the masonry.
d = the depth of water at a given joint x.

$H = \dfrac{d^2}{2r}$ = the horizontal thrust of the water.

$M = \dfrac{d^3}{6r}$ = the moment of H about any point in the joint x.

W = the total weight of masonry resting on the joint x.
w = the total weight of masonry resting on the joint l.
R = the resultant of H and W.
R' = the resultant of the reactions.
α = the angle made by R with a vertical line.
p = the limiting pressure per square foot at the front face of the dam.
q = the limiting pressure per square foot at the back face of the dam.
$q > p$ will be generally assumed.

There are four ways in which a dam may fail:

1st. By overturning.
2d. By crushing.
3d. By sliding or shearing.
4th. By rupture from tension.

To insure ample safety against all these causes of failure, we shall require the profiles which are to be determined to comply with the following conditions:

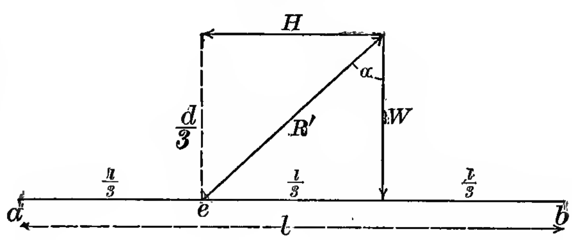
1st. The lines of pressure must lie within the centre third of the profile, whether the reservoir be full or empty.

2d. The maxima pressures in the masonry or on the foundations must never exceed certain fixed limits.

3d. The friction between the dam and its foundation, or between any two parts into which the dam may be divided by a horizontal plane, must be sufficient to prevent sliding.

The first of the above conditions precludes the possibility of tension, and insures also a factor of safety of at least 2 against overturning, as will be seen from the following: Suppose the lines of the reaction R' and of W to intersect the joint l at the limits of its centre third, as shown in Fig. 10. Taking the moments of the forces R', W and H, which are in equilibrium, about the point e, we find

Fig. 10



$$\frac{Hd}{3} = \frac{Wl}{3}.$$

If the moments are taken about the front edge a, the lever-arm of W will be doubled, while that of H remains unchanged. The factor of safety against overturning is therefore 2. It is evident that if the line of action of W or R, or both of them, should intersect l within its centre third, the factor of safety against overturning, called factor of stability, would be greater than 2.

The resistance of a dam to sliding has generally been calculated in the following manner:

If we conceive the wall to be cut by a horizontal plane at a given joint l, there will be two forces which will prevent the upper part from sliding on the lower one:

1st. The cohesion [illegible] the masonry; [illegible]

2d. [illegible]

We must have for equilibrium

$H \overset{=}{<}$ [illegible]

The value of c is considerable for good masonry, but cannot be [illegible]. If we consider cl as a margin of safety, and place

$$H \overset{=}{<} fW, \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad \text{(E)}$$

ample security against sliding will be insured. These formulæ may also be applied to the base of the dam, in which case $c =$ the adhesion of the base to its foundation and the resistance due to the irregularities of the latter.

The value of f has been taken by different writers from .67 to .75. To find in any given case what value of f would prevent sliding, we have, from (E) and Fig. 10,

$$f = \frac{H}{W} = \tan \alpha. \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad \text{(F)}$$

The maxima pressures in the masonry or on its foundation are to be determined by formula A, B or C. In applying these, however, we have followed Prof. Rankine's method of neglecting the vertical component of the water-pressure. As the early French writers adopted very low limits of pressure (6 kilos. per square centimetre), the up-stream sides of the profiles which they designed had considerable slope. The vertical component of the water-pressure became thus an important factor, which had to be considered in the calculations. Experience has since demonstrated that much higher limits of pressure may be safely adopted, especially for the up-stream side, which, under these circumstances, becomes very steep. By neglecting now the vertical component of the water-pressure, when the back face is nearly vertical, only a trifling error in the direction of safety will be made, as the force which has been omitted adds to the stability of the dam, and diminishes slightly the pressures at the front face by moving P up-stream. There are several additional reasons for adopting this course, which will be explained in Chapter IV., page 26.

It is impossible to express the thickness of a dam at a given joint by a formula which will satisfy at once the three conditions given on page 15. However, it will always be found within the limits of practice that by satisfying the first two of them we have also fulfilled the third. The reason of this fact will be explained in Chapter V. We shall base the thickness of a dam, therefore, upon the first two general conditions.

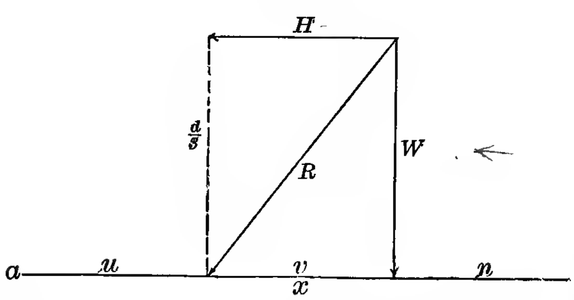
In establishing the necessary equations, we shall consider only one lineal foot of a dam, and suppose it to resist the thrust of the water simply by its own weight. Although we shall imagine this section of a dam to form a rigid monolith of homogeneous masonry, we will assume it for the purposes of calculation to be divided into courses by horizontal planes.

The profiles will be proportioned simply with reference to the hydrostatic pressure of the water, the highest elevation of its surface being taken at the level of the top of the dam. Theoretically, this would allow us to make the top width of the wall equal to zero. However, to resist the action of waves, the shocks from floating bodies, and to serve as a means of communication, a dam must always have more or less top width, which will be determined solely by special considerations, depending upon the locality, etc.

To obtain a profile containing the least area which will fulfil the requirements, we shall calculate the joints to be just on the given limits.

With the conditions advanced in this chapter, a given joint of masonry will always be composed of three parts, as shown in Fig. 11. We can write, therefore,

Fig. 11



$$x = u + v + n; \quad . \quad . \quad . \quad (1)$$

but
$$H\frac{d}{3} = M = Wv,$$

since R is the resultant of H and W.

hence
$$v = \frac{M}{W}. \quad . \quad . \quad . \quad . \quad (G)$$

As we shall adopt polygonal outlines for [illegible] *profile*, each course of

masonry will have a trapezoidal cross-section, and we will have, recollecting that the unit of weight is one cubic foot of masonry,

$$M = v\,W = w + \left(\frac{l+x}{2}\right)h. \quad \ldots \quad \text{(H)}$$

Therefore

$$v = \frac{M}{w + \left(\frac{l+x}{2}\right)h}, \quad \ldots \quad \text{(I)}$$

which value being substituted in Equation (I) gives

$$x = u + \frac{M}{w + \left(\frac{l+x}{2}\right)h} + n. \quad \ldots \quad \text{(II)}$$

By substituting for u and n their proper values, which result from the conditions imposed, we can determine by means of Equation (II) the exact theoretical thickness of a dam at intervals, taken as small as may be desired.

In the upper portions of the profile, where the pressure in the masonry is inconsiderable, u and n will be determined solely by the 1st condition, according to which $u \underset{>}{=} \frac{x}{3}$ and $n \underset{>}{=} \frac{x}{3}$. When, however, the pressures in the masonry reach the limits p and q, the maxima pressures at the joints to be calculated must be kept on these limits by substituting values for u and n derived from formula A (page 10), viz.:

$$u = \frac{2x}{3} - \frac{px^2}{6W}, \quad \ldots \quad \text{(J)} \qquad\qquad n = \frac{2x}{3} - \frac{qx^2}{6W}, \quad \ldots \quad \text{(K)}$$

in which

$$W = w + \left(\frac{l+x}{2}\right)h. \quad \ldots \quad \text{(H)}$$

As we shall commence the design of a dam with a given top width, determined by special considerations, the upper portions of the profile will necessarily have an excess of material as regards resistance to the hydrostatic pressure of the water. To obtain the minimum area of profile, both faces must be continued vertical until one of the limiting conditions is reached. Overhanging faces would give a smaller profile, but should not be adopted for obvious reasons.

The upper portion of the profile ought, therefore, to be a rectangle. P' will pass through its centre, but, owing to the water-pressure, P will gradually approach the front face, reaching eventually a joint $x = a$, where $u = \frac{a}{3}$.

To find the depth of this joint below the top of a dam, substitute in Equation (II)

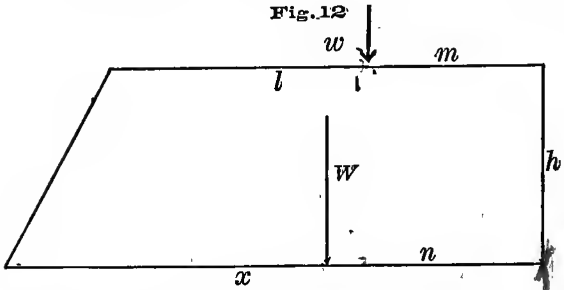
$$x = l = a, \qquad u = \frac{a}{3}, \qquad n = \frac{a}{2}, \qquad h = d, \qquad w = 0.$$

By reducing, we obtain

$$d = a\sqrt{r}. \quad \ldots \quad (1)$$

The front face must now be sloped to keep P on the limit of the centre third of the profile; but, as P' is in the centre of the joint just found, the back face may be continued vertical for a series of courses. Fig. 12 shows the cross-section of one of them.

Fig. 12



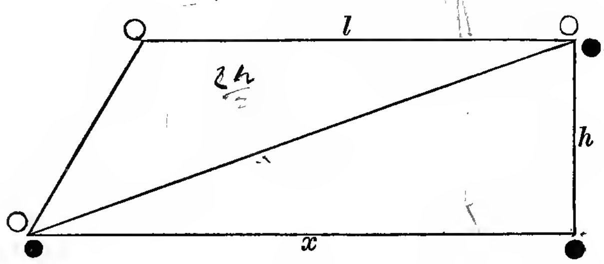
We shall again employ Equation (II) to find x, placing $u = \frac{x}{3}$. As P' will lie within the centre third of x, n will not be determined by the limiting conditions, but can be found by taking moments about the back edge of the course.

The moment of $W = Wn$.

The moment of $w = wm$.

To find the moment of the trapezoid $\left(\frac{x+l}{2}\right)h$, divide it into two triangles, as shown in Fig. 13.

Fig. 13



$$\circ = \frac{lh}{6}.$$

$$\bullet = \frac{xh}{6}.$$

As the centre of gravity of a triangle equals the centre of gravity of three equal weights placed at its apices, we may substitute for the trapezoid 6 weights, as show in the figure. We will find

$$\text{moment of } \left(\frac{x+l}{2}\right)h = (x^2 + lx + l^2)\frac{h}{6}.$$

The moment of the whole weight must equal the moments of its parts; therefore

$$Wn = wm + (x^2 + lx + l^2)\frac{h}{6};$$

whence, placing

$$W = w + \left(\frac{x+l}{2}\right)h, \quad \ldots \ldots \ldots \quad \text{(H)}$$

we obtain

$$n = \frac{(x^2 + lx + l^2)\frac{h}{6} + wm}{w + \left(\frac{x+l}{2}\right)h}.$$

Substituting this value and also $u = \frac{x}{3}$ in Equation (II), we find, after reducing,

$$x^2 + \left(\frac{4w}{h} + l\right)x = \frac{6}{h}(wm + M) + l^2. \quad \ldots \ldots \ldots \quad (2)$$

Equation (2) may be used for a series of joints, until one is found where $n = \frac{x}{3}$.

For the next course both faces will have to be sloped so that $u = n = \frac{x}{3}$. Substituting these values in Equation (II), and reducing, we obtain

$$x^2 + x\left(\frac{2w}{h} + l\right) = \frac{6M}{h}. \quad (3)$$

Thus far we have only been obliged to introduce the first general condition (page 15) into our formulæ, as the pressures in the upper portions of the profile do not reach the limit. However, in applying Equation (3) to a series of joints, we must always, after finding a value for x, calculate the maxima pressures, both with the reservoir full and empty, to see whether they reach the limit p or q. This will always occur first at the front face, as we have assumed $p < q$. When the limit p is reached, the next series of joints must be found by substituting in Equation (II)

$$u = \frac{2x}{3} - \frac{px^2}{6\left(w + \left(\frac{x+l}{2}\right)h\right)}, \quad (J) \qquad n = \frac{x}{3};$$

after reducing, we obtain

$$x^2 = \frac{6M}{p}. \quad (4)$$

This equation may be employed until the limiting pressure is reached at the back face. We must then determine the next joints by substituting in Equation (II)

$$u = \frac{2x}{3} - \frac{px^2}{6\left(w + \frac{(x+l)h}{2}\right)}; \quad (J)$$

$$n = \frac{2x}{3} - \frac{qx^2}{6\left(w + \frac{(x+l)h}{2}\right)}. \quad (K)$$

After reduction we find

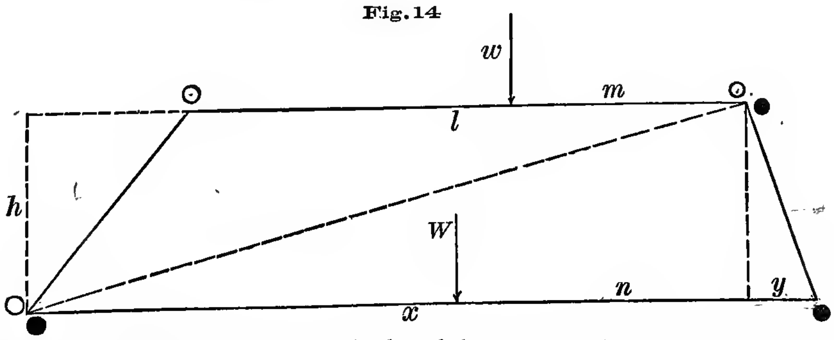
$$x^2(p + q - h) - 2x\left(w + \frac{lh}{2}\right) = 6M. \quad (5)$$

Equations (1) to (5) enable us to calculate successively the exact lengths of all joints, commencing at the top of the dam. However, Equations (3), (4) and (5), which apply to that portion of the dam where both faces slope, determine only the lengths of the joints, but not their position. This may be found by the following equations, which determine the batter of the back face of the given course:

Fig. 14

$$\circ = \frac{lh}{6};$$

$$\bullet = \frac{xh}{6}.$$



In Fig. 14, w, m, l, h and x, which has been calculated by one of the given equations,

are known. The quantity to be found is y, the batter of the back face. The trapezoid $\frac{(l+x)h}{2}$ will be replaced by six weights, as already explained in finding Equation (2). As the moment of the whole weight must equal the sum of the moments of its parts, we have, taking moments about the back edge of x,

$$Wn = w(m+y) + \frac{h}{6}(l+x)y + \frac{h}{6}(l+x)x + \frac{h}{6}(l+y)l. \quad . \quad . \quad . \quad . \quad (\text{III})$$

For Equations (3) and (4) the value found for y must make $n = \frac{x}{3}$.

Substituting this value in the above equation, and also

$$W = w + \left(\frac{l+x}{2}\right)h, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{H})$$

we shall find, after reducing,

$$y = \frac{2w(x-3m) - hl^2}{6w + h(2l+x)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

For Equation (5) we must have

$$n = \frac{2x}{3} - \frac{qx^2}{6\left[w + \left(\frac{l+x}{2}\right)h\right]}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{K})$$

Substituting this value in Equation (III) we obtain, after reduction,

$$y = \frac{w(4x-6m) + lh(x-l) + x^2(h-q)}{6w + h(2l+x)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Equations (1) to (5) are simply modifications of the general Equation (I), resulting from the changes in the controlling influence of the limiting conditions in the various parts of the dam. In the upper portions of the profile, the limiting of the positions of the lines of pressure to the centre third of the profile is the only important condition, whereas in the lower portions the amount of pressure in the masonry becomes the controlling consideration. What adds to the mathematical difficulties of finding simple equations for determining the proper profile is the fact that the changes in the controlling influence of the conditions do not occur simultaneously at the front and back face.

Feeling convinced that with such complicated requirements no regular mathematical curves or lines could be found for bounding a profile which should fulfil all the given conditions and at the same time involve a minimum amount of masonry, the writer, instead of following the methods hitherto proposed for determining at once a practical profile, adopted the plan of first finding by means of the equations given a *theoretical profile* upon which to base the practical design. The *theoretical profile* resulting from calculating the required thickness of a dam at regular intervals will have polygonal faces. By taking the value of h sufficiently small, we can determine a profile which shall fulfil all the given conditions and at the same time contain practically the minimum area. The only modification that remains to be made in this theoretical type to arrive at the practical design is to simplify its faces by fitting curves or straight lines to the theoretical form. The effect of small changes, made for this purpose, will be but slight as regards the given conditions.

In applying the equations to practical calculations we have assumed $h = 10$ feet, as the numerical work is thereby greatly facilitated, and the transition from one equation to the other is so gradual that no difficulty is experienced in selecting the proper one.*

The equations given thus far have been based upon the conditions of Prof. Rankine. *They are the only ones that we shall advance for designing profiles.* For sake of comparison, however, we shall show how Equation (11) may be adapted to the French conditions as given by MM. de Sazilly, Delocre and Pelletreau, with the difference that, for the reasons stated on page 16, the vertical component of the water-pressure will be omitted. The main difference between the principles adopted by these engineers and those of Prof. Rankine is, that in the former no restriction is placed upon the positions of the lines of pressure. So long as low limits of pressure are used no danger will result from this neglect, as P and P' will lie practically within the centre third of the profile. But when we take high limits of pressure—and the tendency of late has been in this direction—the lines of pressure will be very eccentric. With such limits the back face of a dam will be vertical for a great depth, and the effect of neglecting the vertical component of the water-pressure is therefore slight. The profile may be found by the following method:

The upper portion of the profile will be a rectangle, but instead of terminating where P reaches the limit of the centre third of a joint, it will be continued until the pressure at the front face reaches the limit p. Equation (1), $x = u + v + n$, is to be used with the following substitutions:

$$n = \frac{a}{2}; \qquad v = \frac{M}{W} = \frac{d^2}{6ra};$$

$$u = \frac{2a}{3} - \frac{pa^2}{6W} \text{ (from formula } A\text{) if } u > \frac{a}{3},$$

$$u = \frac{2W}{3p} \text{ (from formula } C\text{) if } u < \frac{a}{3},$$

in which $W = ad$.

If $u > \frac{a}{3}$, we obtain

$$d^3 + ra^2d - pra^2 = 0. \qquad (8)$$

If $u < \frac{a}{3}$, we will find

$$d^2 + \frac{4ra^2d}{p} = 3ra^2. \qquad (9)$$

As we do not know *à priori* whether u will be greater or less than $\frac{a}{3}$, we can only determine by trial which Equation, (8) or (9), ought to be used.

The limiting pressure will evidently be reached sooner at the front than at the back face. The latter may therefore remain vertical for the next series of joints, whereas the former must be sloped to keep the maxima pressures equal to p. Substituting in Equation (11):

$$u = \frac{2}{3}x - \frac{px^2}{6\left(w + \left(\frac{l+x}{2}\right)h\right)} \text{ (J)} \qquad \text{if } u > \frac{x}{3},$$

* A practical example of the use of equations (1) to (5) is given in the Appendix, page 235.

$$u = \frac{2\left(w + \left(\frac{l+x}{2}\right)h\right)}{3p} \text{ (from formulæ C and H)} \qquad \text{if } u < \frac{x}{3},$$

and for n the same value found for Equation (2) by taking moments around the back edge of the joint, we obtain—

If $u > \frac{x}{3}$,

$$px^2 + 2wx = 6wm + hl^2 + 6M. \quad \ldots \ldots \ldots \quad (10)$$

If $u < \frac{x}{3}$,

$$x^2\left(h - \frac{h^2}{2p}\right) + x\left(3w + lh - \frac{1}{p}(2hw + lh^2)\right) = 3M + 3wm + \frac{l^2h}{2} + \frac{1}{p}\left(2w^2 + 2wlh + \frac{h^2l^2}{2}\right). \quad (11)$$

As in the previous case, we can only determine by trial which of the above equations to use.

So soon as the limiting pressure has been reached at the back face, both faces of the dam will have to be sloped in order to keep the maxima pressures within the prescribed limits. Sazilly, Delocre and Pelletreau used the same limit of pressure for both faces; we shall, however, introduce p and q as in Equations (1) to (5). Let us first suppose that P lies within the centre third of a joint x, while P' lies without. Substituting in Equation (11)

$$u = \frac{2x}{3} - \frac{px^2}{6\left(w + \left(\frac{l+h}{2}\right)h\right)} \text{ (J)},$$

$$n = \frac{2\left(w + \left(\frac{l+x}{2}\right)h\right)}{3q} \text{ (from formulæ C and H)},$$

we obtain

$$x^2(hq + qp - h^2) + x[(q - 2h)(2w + lh)] = 6qM + lh(lh + 4w) + 4w^2. \quad \ldots \quad (12)$$

Should P lie without and P' within the centre third of a joint, we must employ Equation (12), transposing simply the position of p and q.

If both P and P' are within the centre third of the joint, we obtain Equation (5), given on page 19.

Finally, if P and P' both are outside of the centre third, we obtain

$$x^2\left[\frac{h}{2}(3 - hz)\right] + x\left[3w + h\left(\frac{3l}{2} - lzh - 2wz\right)\right] = z\left[2w^2 + hl\left(\frac{hl}{2} + 2w\right)\right] + 3M, \quad (13)$$

in which $z = \frac{p+q}{pq}$.

The above equations cover all the cases that can arise, but some of them will never be needed for actual calculations.

The value of y will be found by using Equation (6) or (7), except when P' lies outside of the centre third of the joint. In this case

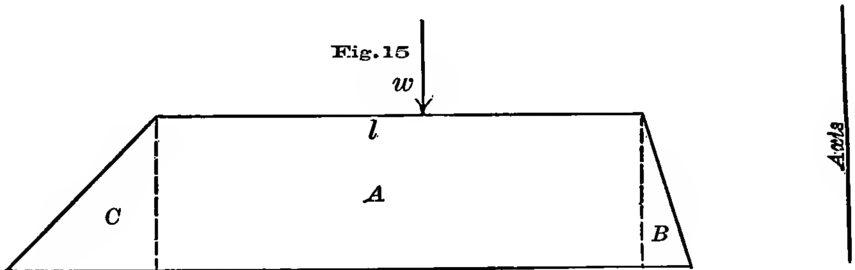
$$y = \frac{\frac{4W^2}{q} - 6wm - h(x^2 + xl + l^2)}{6w + h(2l + x)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (14)$$

In the French methods of determining profiles, in which P and P' are not restricted to its centre third, there are always more or less trial-calculations involved, as it is not known in advance whether to employ formula A or C for expressing u or n. However, if we assume $h = 10$ feet, the change from one equation to the other is so gradual that trial-calculations will seldom have to be employed.

In applying the equations given in this chapter to practical examples, it is advisable to check the calculations by some other method. This may be done readily by the principle of moments, as follows:

Assume a vertical axis at a convenient distance from the back face. After having found a value for a joint x by solving the proper equation, divide the corresponding course into a rectangle and one or more triangles by vertical lines, as shown in Fig. 15.

Fig. 15



The moment of the weight w resting on the joint l about the axis is supposed to be known from the previous "check-calculation." Adding the weights of A, B and C (the parts into which the course has been divided), expressed in cubic feet of masonry, to the weight w, and the moments of A, B and C about the vertical axis to the moment of w, we obtain W (the total weight resting on the joint x) and its moment about the axis. By dividing this moment by the weight W, we obtain the distance of the centre of gravity of the profile above the joint x from the axis; in other words, we have found where P' (the line of pressure, reservoir empty) cuts the joint x.

From the formula (G), $v = \frac{M}{W}$, in which $\begin{cases} M = \text{moment of water,} \\ W = \text{total weight,} \end{cases}$

we find the distance from P' to the line of pressure P (reservoir full), measured on x. We can verify thus whether P and P' are within the prescribed limits. By means of formulæ A, B and C the maxima pressures at the joint x can be found, both for reservoir full and empty. These pressures should be within the given limits.

The value of the coefficient of friction which is necessary to prevent sliding can be found from (F) and Fig. 10 as follows:

$$f = \tan \alpha = \frac{v}{\frac{d}{3}} = \frac{3v}{d}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (L)$$

The factor of stability, which we shall denote by S, can be obtained thus:

$$S = \frac{W(u + v)}{M} \text{ (see page 15), but } M = vW \text{ (from G); hence } \quad S = \frac{u + v}{v}. \quad . \quad . \qquad (M)$$

CHAPTER IV.

VARIOUS APPLICATIONS OF EQUATIONS (1) TO (14).

In the preceding chapter we have found the necessary equations for calculating *theoretical profiles*. We will now apply them to practical examples from which important deductions may be made.

Necessity of Limiting the Positions of the Lines of Pressure.—The maxima pressures in existing dams vary from 6 to 14 kilos. per square centimetre (about 6 to 14 tons per square foot), as may be seen in Table XXIII. The limit adopted for the Furens Dam was 6½ kilos. per square centimetre; for the Ban Dam, built subsequently, it was raised to 8 kilos.; and M. Græff in his memoir* on the former structure gives a profile based upon 14 kilos. per square centimetre, which pressure has been safely sustained in the Almanza Dam for three centuries.

To study the effect of adopting the French conditions (see page 21) with high limits of pressure, we give in Plates VIII. and IX. two profiles calculated by Equations (8) to (14), assuming for the first,

$$p = 8 \text{ kilos. per square centimetre,}$$
$$q = 10 \text{ kilos. per square centimetre;}$$

and for the second,

$$p = q = 14 \text{ kilos. per square centimetre.}$$

Tables VIII. and IX. give the necessary details.

The danger resulting from placing no limits to the positions of the lines of pressure is very apparent in these profiles; especially in No. 2, which corresponds to the one given by M. Græff for a limiting pressure of 14 kilos. per square centimetre. In this case the lines of pressure are so eccentric in the upper part of the profile, that a dam built according to this design would be very unsafe, and by no means have a "profile of equal resistance."

Although experience proves that pressures as great as 14 kilos. per square centimetre may be safely sustained in the lower portions of a dam, where the lines of pressure generally lie within the centre third of the profile; yet in the upper portions such pressures cannot be permitted, as they can only result from a dangerous eccentricity of one of the lines of pressure. We see, therefore, the necessity of adopting some principle besides the limiting of the stress on the masonry, in order to insure perfect safety in a dam.

If we add to the French conditions the one given by Prof. Rankine, which limits the positions of the lines of pressure to the centre third of the profile, no danger can

* In the "Annales des Ponts et Chaussées," Sept. 1866.

arise in the upper portions of a dam, designed on this basis, on account of a high limit of pressure being assumed. Such a wall would always have a factor of safety against overturning of at least 2; would have no part subject to tension; and would also have ample safety against sliding or shearing, as will be shown in the next chapter.

Weight of Masonry.—The early writers on the subject of dams assumed the specific gravity of the masonry as 2. M. Krantz, in his book on "Reservoir Walls,"* points out that this assumption introduces an error into the calculations, as the true specific gravity of good rubble masonry built of granite or limestone is about 2.3. M. Bouvier † gives the specific gravity of the masonry in the Ternay Dam, constructed of granite, as 2.36; M. Pochet ‡ places that of the Habra Dam, which was built of calcareous stone, at 2.15.

To show the influence of the weight of the masonry upon the form of the profile, we have calculated four profiles by Equations (1) to (7), assuming the specific gravity of the masonry respectively at 2, $2\frac{1}{6}$, $2\frac{1}{3}$, and $2\frac{1}{2}$. The corresponding values of the moments of water (M) are:

$$\frac{d^3}{12}, \qquad \frac{d^3}{13}, \qquad \frac{d^3}{14}, \qquad \frac{d^3}{15},$$

the weights per cubic foot of masonry being 125, 135.41, 145.83, 156.25 lbs.

As the exact average weight of the masonry in a dam is generally unknown, we may adopt approximate whole numbers as the divisors of M, in order to facilitate the numerical work.

The four profiles are shown in Plate X. Tables X., XI., XII. and XIII. give the details.

If we compare the corresponding areas of these profiles at different depths, we shall find that for a depth of about 190 feet the area of profile diminishes as the weight of the masonry increases. For greater depths, however, this law will be reversed.

Not only will the amount of masonry required for a dam depend upon the specific gravity of the masonry, but this factor will also affect the form of profile, as will be noticed in Plate X.

Vertical Component of the Water-pressure.—Let us next examine the effect of including the vertical component of the water-pressure in our calculations. Taking profile No. 5 (Table XII.) and recalculating the maxima pressures at the front face, including the vertical component of the water-pressure in the total weight resting on a joint, we find as results the figures given in the table at the top of the next page.

This table shows that although by including the vertical component of the water-pressure a greater load has been placed upon each joint, yet, owing to the fact that the line of pressure has been moved back from the front face of the dam, the maxima pressures will be diminished.

The difference between the pressures calculated with and without the vertical component of the water-pressure increases gradually, amounting to 15 per cent at a depth of 200 feet. Now, this gradual reduction of the amount of pressure at the front face as its

* Published in Paris in 1870.

† "Annales des Ponts et Chaussées," Aug. 1875.

‡ Ibid., April 1875.

1 Depth of Water. Feet.	Maximum Pressure. 2 Vertical Component of Water included in Cubic Feet of Masonry.	 3 Vertical Component of Water excluded in Cubic Feet of Masonry.	4 Numbers in Column 3 divided by Corresponding Numbers in Column 2.
70	81.26	82.5	101
80	86.31	88.4	102
90	94.09	96.2	102
100	102.16	104.4	102
110	110.10	112.3	102
120	108.91	112.3	103
130	108.23	112.3	104
140	107.54	112.3	104
150	106.96	112.3	105
160	106.55	112.3	105
170	103.88	112.3	108
180	101.77	112.3	110
190	98.00	112.3	114
200	97.22	112.3	115

batter increases is precisely what one of Prof. Rankine's conditions requires. As this eminent writer admits that it is impossible, in our present state of knowledge, to find the law this diminution of stress ought to follow, it would seem sufficient to effect this object by simply omitting the vertical component of the water-pressure in the calculations.

There is another reason for adopting the above method. Formulæ A, B and C (page 10) have been obtained by assuming a dam to be perfectly rigid. It has been pointed out in Chapter II. that we must use caution in applying these formulæ, which after all are only approximately true, to extreme cases. Now by assuming in the case of a dam having a steep back face that a column of water resting near one edge of a long joint relieves the pressure at the other edge, we are carrying the hypothesis of rigidity to an unsafe extreme. It is certainly safer to overestimate than to underestimate the maxima pressures.

There is no theoretical difficulty in modifying the equations already given so as to include the vertical component of the water-pressure; but we shall obtain an equation of the fourth degree of such length as to be of no practical value. Various approximate methods of finding the desired result can readily be found. Thus Equations (1) to (5) may be used with the value of p increasing gradually below the joint, where the back face commences to slope, the increase being based upon the above table. When the pressures are afterwards recalculated with the vertical component of the water-pressure included in the loads, they will be found to be very near the fixed limit. The reasons given above show the advisability of neglecting the vertical component of the water-pressure, where

the up-stream face of a dam is steep, as any error that may result therefrom will be on the safe side; whereas by including the component in the equations we probably err in the other direction.

Inclined Joints.—In making calculations for the profile of a dam, it is customary to assume it to be divided into courses by horizontal joints. As these are imaginary, there is no theoretical reason why they should be assumed to be horizontal. The question naturally arises, what the effect would be of making calculations for inclined joints. In Plate XI., profile No. 5 is shown with joints radiating from points in the back face of the wall. By examining Table XIV., which gives the results of the calculations made for these joints, it will be seen that by inclining them downward from the back of the dam the maxima pressures are reduced; whereas by inclining them upward the opposite effect will be produced within certain limits. The angles made with a horizontal line which give the maxima pressures at a certain depth of water vary from 20°–30°. The increase of pressure resulting from inclining the joints, which at a depth of water from 60—110 feet amounts to only 14%, is 41% at a depth of 160 feet. In making these calculations the weight of the masonry below the joints has been omitted, as though the dam were cut in two parts. While this supposition is rather extreme, good rubble masonry would doubtless bear the resulting pressures.

Should we desire to increase the thickness of a dam on account of the pressures produced upon oblique joints, we can modify the Equations (1) to (14) according to M. Bouvier's method,* so that the limiting pressure will not be exceeded in joints drawn perpendicular to the resultant pressure.

Profile on Bouvier's Principle.—Let us change profile No. 5 in accordance with the above. No alteration is necessary for the first 90 feet from the top of the dam, as the only important consideration for this part is that the lines of pressure must be within the centre third of the profile. Below this depth, as the pressures on the joints will be parallel with the resultant, we can take q for the limiting pressure at both faces. The corresponding limit of vertical pressure at the front face will be $q \cos \alpha$, in which α is the angle which the resultant (R) makes with a vertical line. The maxima pressures at the back face are of course calculated for horizontal joints, as they occur, when the reservoir is empty and R, therefore, vertical. To make Equations (1) to (14) agree with M. Bouvier's principle we need only substitute for p the pressure $q \cos \alpha$. The angle (α) is unknown, as its value cannot be determined until we have found the corresponding joint. But the difference in the inclination of the resultants from joint to joint is so slight, when these are only ten feet apart, that we can use the value of α for the joint above the one to be determined, in the equations. This is the method we have followed in modifying profile No. 5 to agree with Bouvier's principle. The profile obtained thus is shown in Plate XII., the details being given in Table XV. By examining this table it will be seen how trifling the differences are in the value of (α) from joint to joint. The bottom width of the profile found by Bouvier's principle is 196.35 ft., and its area 15,662 sq. ft.; whereas for profile No. 5 we have respectively 190.98 ft. width and 15,157 sq. ft. area.

Although we have given this method for taking the obliquity of the pressures, when the reservoir is full, into account, our present knowledge of this subject is so uncertain

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* See page 12.

that it is a useless refinement to introduce it into the equations. For all practical purposes we need only consider horizontal joints, applying Equations (1) to (7) with such values for p and q as experience warrants.

Comparison of a Theoretical Profile with Rankine's Logarithmic Type.—Equations (1) to (5) have been based upon the conditions given by Prof. Rankine. The *theoretical profile* found by applying them differs, however, considerably from the logarithmic profile designed by that eminent engineer. For sake of comparison we have made calculations for a *theoretical profile* based upon the data used by Rankine. Plate IV. shows the profile and also the logarithmic type. Tables III. and IV. give the details.

For a depth of 140 feet both profiles are based upon exactly the same conditions and data; but below this depth there is a slight difference, which requires explanation. Prof. Rankine states, namely, in the report we have quoted on page 4, that the limiting vertical pressure should be diminished as the batter of the front face increases. He did not advance any law for this diminution, but simply designed his profile type in such a manner that the stress at the outer face at a depth of 160 feet would equal the pressure sustained in the Furens Dam at the same depth, 6½ kilos. per square centimetre. Having adjusted the logarithmic curves, adopted for the outlines of his design, with reference to this stress, and also to keep the lines of pressure practically within the centre third of the profile, Rankine was unable to regulate the pressures in the lower portions of the profile, as they are determined without any regard to theory by the logarithmic curves. He considered it an advantage that the outlines chosen for his profile-type made the pressures diminish rapidly at the front face of the lower portions of the dam; but if his principle be correct, the steeper the front face is kept within safe limits of pressure, the better.

In our *theoretical profile* we have retained the same limit of pressure for all parts of the dam, the result being shown in the following table:

Depth.	Rankine's Logarithmic Profile.		Theoretical Profile.	
Feet.	Angle.	Pressure in Feet of Masonry.	Angle.	Pressure in Feet of Masonry.
100	52° 47′	122	54° 14′	103
110	49° 17′	124	54° 21′	112
120	45° 44′	123	54° 30′	122
130	42° 09′	119	48° 56′	125
140	38° 37′	114	45° 21′	125
150	35° 11′	107	44° 07′	125
160	31° 41′	99	42° 59′	125
170	28° 46′	90	41° 57′	125
180	25° 51′	81	40° 59′	125
190	23° 19′	73	39° 13′	125
200	20° 40′	64	38° 09′	125

The angles given above are made by the tangents of the front logarithmic curve and the lines forming the front face of the *theoretical profile*, respectively, with horizontal planes.

As the profile calculated by our equations has a steeper face than the logarithmic type, it can safely sustain greater pressures. At a depth of 180 feet the difference in the pressures in the two profiles for the same angles is about 8 per cent; at a depth of 200 feet it is less than 10 per cent. As the actual pressures will be somewhat reduced from what is given in the table by the vertical component of the water-pressure, which has been omitted in the calculations, there seems no necessity of reducing the limit of pressure in the lower portions of the dam.

Rankine gives the logarithmic profile only for 180 feet depth of water. If it be continued, the front face becomes very flat. Instead of such a profile having great strains near the front face at the base, it is much more likely that the thin toe of masonry at the front face transmits but little pressure, the stresses following short direct lines towards the base.

The *theoretical profile* agrees for a depth of 140 feet exactly with the conditions and data assumed by Prof. Rankine for his logarithmic profile, and the differences below the depth are but trifling. It will, therefore, be interesting to compare the corresponding areas of the two profiles (see Tables III. and IV.). The following comparison shows that the differences are always in favor of the *theoretical profile*:

Depth.	Rankine's Logarithmic Profile. Area in Square Feet.	Theoretical Profile. Area in Square Feet.	Differences in Favor of the Theoretical Profile.
0	0	0	0
10	200	187	13
20	426	374	52
30	679	565	114
40	973	792	181
50	1,303	1,074	229
60	1,674	1,419	255
70	2,098	1,842	256
80	2,577	2,347	230
90	3,119	2,930	189
100	3,734	3,589	145
110	4,431	4,322	109
120	5,221	5,129	92
130	6,116	6,018	98
140	7,129	7,011	118
150	8,278	8,116	162
160	9,581	9,337	244
170	11,055	10,678	377
180	12,728	12,142	586
190	14,621	13,746	875
200	16,765	15,510	1,255

From Plate IV., and from what has been said above, it will be seen that while the logarithmic profile has sufficient strength and graceful outlines, it is not a close approximation to the correct theoretical form.

In the next chapter we shall show how the *theoretical profiles* calculated by the equations we have given may form the basis of practical designs for masonry dams.

CHAPTER V.

PRACTICAL PROFILES.

THE practical profiles for masonry dams, which we shall establish in this chapter, will be based upon theoretical types containing the least areas consistent with the following conditions:

1st. The lines of pressure must lie within the centre third of the profile, whether the reservoir be full or empty.

2d. The maxima pressures in the masonry or on the foundation must not exceed certain safe limits.

3d. The friction between the dam and its foundation, or between any two parts into which the wall may be divided by a horizontal plane, must be sufficient to prevent sliding.

To these conditions, in which only the hydrostatic pressure of the water is considered, we must add:

4th. The dam must be sufficiently thick in all parts to resist the action of waves and shocks from floating bodies.

As what constitutes sufficient strength with reference to the last of the above conditions is a matter of judgment, in our present state of knowledge, we shall first determine the correct form of profile as regards the first three conditions, and modify it subsequently to satisfy the fourth.

The width of this profile at the highest elevation of the water-surface should evidently be zero. In the upper part of a dam the pressures in the masonry are so inconsiderable, that only conditions 1 and 3 need be considered in proportioning the profile. Within the limits of practice, however, a profile based upon the former condition will always satisfy the latter, as will presently be shown.

The profile which contains the minimum area consistent with condition 1 forms a right-angled triangle having its vertical side up-stream. As in such a section the centre of gravity of the area of the profile above any joint lies in a vertical line passing through the up-stream limit of the centre third of this joint, it follows that the line of pressure P' (reservoir empty) will limit the centre third of the profile up-stream.

Denote the base of the triangular profile by x. By changing its length the value of $\frac{u}{x}$ (see page 14) may be made to vary within the limits 0 and $\frac{2}{3}$.

In Equation (1),

$$x = u + v + n = u + \frac{M}{W} + n \text{ (see page 16)},$$

let us substitute

$$u = \frac{x}{3}, \qquad n = \frac{x}{3}, \qquad M = \frac{d^3}{6r}, \qquad W = \frac{dx}{2}.$$

We shall obtain, by reducing,

$$x = \frac{d}{\sqrt{r}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

As x is proportional to d, the line of pressure P (reservoir full) will cut all horizontal joints in like manner as the base, and will form thus the down-stream limit of the centre third of the profile. We conclude, therefore, that the triangular profile whose base is given by Equation (15) has the minimum area consistent with condition 1.

Now let us investigate whether it fulfils condition 3. We have, from Equation (F), page 15,

$$f = \frac{H}{W} = \tan\alpha; \qquad \text{but } H = \frac{d^2}{2r}, \qquad W = \frac{xd}{2} = \frac{d^2}{2\sqrt{r}}.$$

Substituting these values in Equation (F), we find

$$f = \frac{1}{\sqrt{r}} = \tan\alpha. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Let β = the angle between the faces of the triangular profile:

$$\tan\beta = \frac{x}{d}.$$

Substituting for x its value given in Equation (15), we obtain

$$\tan\beta = \frac{1}{\sqrt{r}}.$$

Hence we have

$$f = \tan\alpha = \tan\beta = \frac{1}{\sqrt{r}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

The coefficient of friction necessary to prevent sliding equals, therefore, the tangent of the angle at the top of the triangular profile. Taking $r = 2$ and $r = 3$ as the extreme limits of the specific gravity of the masonry that may occur, we find f to vary between 0.707 and 0.577. M. Krantz and other authorities place the limiting value of f at 0.75. The triangular profile satisfies, therefore, condition 3, and may be continued until the limit of pressure is reached.

The maximum pressure at any joint of this profile (reservoir full or empty) is given by formula B, page 10, according to which $p = \frac{2W}{x}$. As $W = \frac{dx}{2}$, we have

$$p = d. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

The depth of any joint below the surface of the water expresses, therefore, the maximum pressure in that joint in feet of masonry, whether the reservoir be full or empty.

When the limiting pressure has been reached, the triangular profile must terminate, and be continued by means of Equations (4) to (7) (pages 19 and 20).

Plate XIII. shows a triangular profile for a value of $r = 2\frac{1}{3}$, which we shall call Theo-

retical Type No. I. Table XVI. gives the necessary dimensions, etc. It is sufficiently strong in every respect to resist the hydrostatic pressure of the water; but to resist the action of waves, as required by the fourth of the given conditions, it must be modified.

The first step will evidently be to give it sufficient width at the top to resist the action of waves and shocks from floating bodies. If the dam is also to serve as a bridge this width may be still more increased, and will generally be between the limits of 2 to 20 feet.

The top of the wall ought also to be raised a certain height above the highest probable elevation of the water-surface. As the height of the waves will depend largely upon the extent of the reservoir and the depth of the water, high dams ought generally to have a greater superelevation above the highest water-surface than low ones, and ought also to have a greater top width.

The following table, taken from M. Krantz's book on "Reservoir Walls," and M. Crugnola's work on "Retaining Walls and Dams," gives the top widths and superelevation above the water-surface recommended by these engineers:

Depth of Water, in Metres.	Top Width of Dam, in Metres.		Top of Dam above Water, in Metres.	
	Krantz.	Crugnola.	Krantz.	Crugnola.
5	2.00	1.70	0.50	0.50
10	2.50	2.00	1.00	0.90
15	3.00	2.30	1.50	1.30
20	3.50	2.50	2.00	1.50
25	4.00	3.00	2.50	2.00
30	4.50	3.50	3.00	2.40
35	5.00	4.00	3.50	2.80
40	5.00	4.25	3.50	3.00
45	5.00	4.50	3.50	3.25
50	5.00	4.75	3.50	3.50

A good rule for ordinary cases is to make the top width of the dam and the superelevation of its crown above the highest water-surface one tenth the height of the dam, limiting the former to a minimum value of 5 feet, and the latter to a maximum value of 10 feet.

While it is always advisable to provide a reservoir with an overflow-weir sufficiently large to prevent the water from passing over the dam, yet the safest course in designing the profile will always be to assume the level of the water at the top of the dam. Greater freshets may occur than those upon which the size of the waste-weir was based; or, a great demand for water may lead the owners of the reservoir to raise the level of the overflow. In all the calculations for this book, the water-surface has been assumed at the top of the dam.

We shall now investigate what the effect will be of giving a certain top width to the triangular profile. The upper part of the wall will now evidently have a surplus strength with reference to the first three conditions. As this increase is only required near the top of the dam, we shall seek to reach the triangular profile in the shortest practical manner

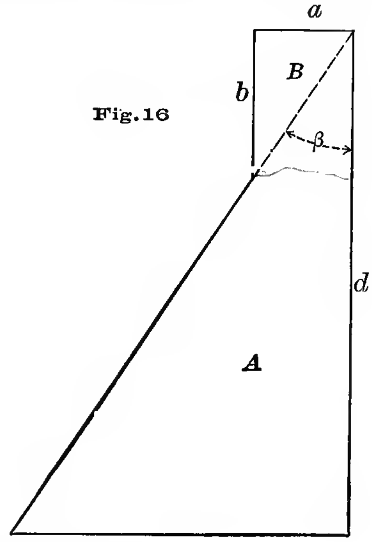
by making the front face vertical until it intersects the sloping face of this profile. Plate XIV. shows the new design, which we shall call Practical Type No. 1.

Let us determine to what extent the positions of the lines of pressure P and P' will be changed.

In Fig. 16 let

a = top width of dam;
b = length of vertical part of front face;
A = triangular profile at a given depth;
B = triangle added at the top of A;
β = angle between the faces of A.

Fig. 16



Using the letters given on page 14, we find:

	Area.	Moment about Back Face.
For A, . . .	$\dfrac{d^2 . \tan\beta}{2}$	$\dfrac{d^3 . \tan^2\beta}{6}$
For B, . . .	$\dfrac{b^2 . \tan\beta}{2}$	$\dfrac{b^3 . \tan^2\beta}{3}$

Hence,
$$n = \frac{\text{Moment of } A + \text{Moment of } B}{\text{Area of } A + \text{Area of } B} = \frac{\tan\beta\,(d^3 + 2b^3)}{3(d^2 + b^2)},$$

and
$$v = \frac{M}{W} = \frac{d^3}{3r(d^2 + b^2)\tan\beta} = \frac{d^3 \tan\beta}{3(d^2 + b^2)} \text{ (from (G) and (17))}.$$

Substituting the above values for n and v in Equation (1), $x = u + v + n$, and recollecting that $x = d . \tan\beta$, we obtain

$$\frac{u}{x} = 1 - \frac{2(d^3 + b^3)}{3(d^3 + db^2)},$$

which expresses the distance of P from the front edge of a joint as a fraction of the length of the joint for any value of $d \geqq b$.

To comply with condition 1 we must have

$$\frac{u}{x} \geqq \frac{1}{3}.$$

If $d = b$, $\dfrac{u}{x} = \dfrac{1}{3}$; if $d > b$, $\dfrac{u}{x} > \dfrac{1}{3}$.

By means of the Differential Calculus we find*

$$\text{Maximum value of } \frac{u}{x} = 0.40392;$$
$$\text{occurring at } d = 1.67765b.$$

Below this depth $\dfrac{u}{x}$ will continually approach the value $\dfrac{1}{3}$, reaching it when d becomes infinite.

* Note A, page 82.

It can easily be shown that when $d < b$, $\frac{u}{x} > \frac{1}{3}$.

Thus we see that for the important case of "reservoir full" the effect of the triangle B is to keep the line P within the centre third of the profile.

Let us now examine its influence on the line P'. Until $d = 2b$, at which depth the centres of gravity of A and B lie in the same vertical line, we shall have $\frac{n}{x} > \frac{1}{3}$. Below this depth, $\frac{n}{x} < \frac{1}{3}$.

Applying the Differential Calculus, we find*

$$\text{Minimum value of } \frac{n}{x} = 0.32218;$$

$$\text{occurring at } d = 3.1038b.$$

From this depth down, the value of $\frac{n}{x}$ will approach $\frac{1}{3}$, reaching it at an infinite distance. The deviation of P' outside of the centre third of the profile is so slight as to be of no practical importance. In examining the profiles designed by Rankine, Harlacher and Crugnola (see Tables III., VI., VII.), we find:

Profile.	Minimum Value of $\frac{n}{x}$.
Rankine,	0.308
Harlacher,	0.316
Crugnola,	0.325

In Prof. Rankine's profile we find that even P is allowed to deviate slightly outside the given limits, so as to give a minimum value of $\frac{u}{x} = 0.308$.

While it is best to confine P strictly within the centre third of the profile, so as to prevent all possibility of tension at the back face of the dam, a slight deviation from the centre third of the profile by P' may be permitted.

With reference to stability and shearing strength, profile A will evidently be improved by the additional weight B. We conclude, therefore, that by adding a section like B, with any top width, to the triangular profile, we obtain at once a Practical Type No. 1 (see Plate XIV.), which may be employed for any height that gives pressures at the base within the given limits. This type fulfils practically the four given conditions, and the only question which remains to be examined is whether it is the most economical profile which can be found.†

To investigate this subject we have calculated Table XVII. for a dam built according to Practical Type No. 1, the top width being assumed as 20 feet. We have also computed Table XVIII. for a theoretical profile having the same top width, and being based simply on the condition that it shall contain the minimum area that will keep the lines P and P' within its centre third. This profile, which we shall call Theoretical

* Note B, page 82.

† A profile of this kind was proposed by Prof. Castigliano in the "Politecnico" for 1884.

Type No. II, will be found in Plate XV., the triangular profile (No. I) being shown by a dotted line for comparison. By comparing profiles I and II by means of Plate XV. and Tables XVI. and XVIII., we see that the effect of the area added at the top of No. II is twofold: 1st. To reduce the thickness of the profile from that of No. I; 2d. To move the lower part of No. II up-stream from the position of I.

As the influence of the wide top of No. II is offset by the reduction of its area lower down, this profile approaches the form of No. I as the depth increases, the corresponding faces becoming nearly parallel.

If we examine the batters of the faces of No. II we find that the front face forms a reverse curve, being first concave down-stream and then up-stream, approaching a line parallel with the front face of type I. The back face of No. II is first concave up-stream and then down-stream, approaching rapidly a vertical line.

Now before we compare No. II with the Practical Type No. 1 we must simplify its outlines to obtain a Practical Type No. 2. Plate XVI. shows this profile, and Plate XV. the theoretical type upon which it is based. Table XIX. gives the necessary dimensions, etc. The straight lines and circular curve which have been substituted for the many changes in the outlines in the theoretical type change that profile but slightly.

To compare the corresponding areas of the Theoretical Types I and II, and of the Practical Types 1 and 2, we have considered the area of No. I at any joint as the unit, and have obtained thus the following table:

Depth of Water, in Feet.	Theoretical Types.		Practical Types.	
	I.	II.	1.	2.
	Area.	Area.	Area.	Area.
0	0	0.000	0.000	0.000
20	1	3.055	3.055	3.055
40	1	1.561	1.583	1.579
60	1	1.186	1.259	1.196
80	1	1.064	1.146	1.080
100	1	1.027	1.093	1.041
120	1	1.013	1.065	1.027
140	1	1.007	1.047	1.022
160	1	1.004	1.037	1.018
180	1	1.002	1.029	1.016
200	1	1.002	1.023	1.015

The above table shows that the differences in the corresponding areas become rapidly less as the depth increases, the profiles being always in the following order as regards smallness of area: I, II, 2, 1. We conclude, therefore, that Practical Type No. 2 is always preferable to No. 1 as regards economy of material. Although the difference between their areas amounts to less than 1 per cent at the base, it is 5.3 per cent at a depth of 60 feet.

The relation between the areas of types I, II, 1 and 2, shown in the table above is general and does not depend upon the top width adopted. For, if we plot these

profiles to any scale, others, having any desired top width, may be obtained by simply changing the scale. The new profiles will always satisfy condition 1.

Thus far we have paid no attention to the pressures in the Practical Types 1 and 2, or to their resistance to sliding or shearing. As regards the pressures, Tables XVII. and XIX. show that they increase gradually in these practical types from the top to the base, where they reach the following maxima values for a height of profile of 200 feet:

	Maxima Pressures. Reservoir full. Tons of 2000 lbs. per sq. ft.	Reservoir empty. Tons of 2000 lbs. per sq. ft.
Practical Type No. 1, ...	14.35	15.16
Practical Type No. 2, ...	14.32	14.65

The greatest of these pressures is but slightly in excess of the 14.33 tons per square foot (14 kilos. per square centimetre) sustained by the masonry of the Almanza Dam for over three hundred years without damage. While such pressures cannot be permitted in the upper part of a dam, where they could only result from a dangerous eccentricity of one of the lines of pressure, they can be sustained safely in the lower part, where the lines of pressure will be within the centre third of the profile. There is another important consideration which makes a gradual reduction of pressure from the top to the base of the dam advisable. The strength of the masonry depends, namely, on that of the mortar, whose resistance to crushing increases for a certain time with its age. A dam is weakest, therefore, when just built, its strength diminishing from the base to the top, where the masonry was laid last.

With respect to sliding or shearing, Practical Types 1 and 2 will have greater strength than the triangular type No. I, as their areas at any joint are respectively greater than that of the latter type, and hence give smaller values for f in the formula (F) (page 15). We have already demonstrated that the triangular type No. I has ample strength against sliding or shearing, and types 1 and 2 are still safer in this respect, as shown above.

When the limit of pressure has been reached in any of the types I, II, 1 or 2, more batter must be given to the faces by applying Equations (4) to (7). In this case the areas of the profiles will be greater than if the above-mentioned types had been continued to a corresponding depth, and consequently their resistance to sliding or shearing will also be increased. It follows, therefore, that so long as the lines of pressure are kept within the centre third of the profile, a dam will always have ample strength against sliding or shearing.

From what has been shown regarding the strength of the Practical Types 1 and 2 we can draw the general conclusion, that the profile of a dam which is to be built of ordinary rubble-masonry, weighing about 145 lbs. per cubic foot, can safely be based upon the first general condition given at the commencement of this chapter, provided its height does not exceed 200 feet. The Furens Dam, which surpasses all other reservoir walls in height, has a maximum elevation of 194 feet above its foundation. Higher dams may be built in the future, but will probably be exceptional. Condition 2, which limits the pressure in the masonry, will, therefore, but rarely have to be considered. As regards condition 3, we have shown that it is necessarily fulfilled by our satisfying condition 1.

The facts stated above make the design of an ordinary masonry dam, having a height

of less than 200 feet, a very simple matter. Standard profiles, similar to Practical Types 1 and 2, can be drawn for different weights of masonry, and can be used for lesser heights than their own (which is assumed as 200 feet) by simply changing the scale of the drawing. Tables giving the dimensions and strength of the derived profiles can be readily obtained from Standard Tables like Nos. XVII. and XIX. by the simple process of division explained in those tables. It can easily be proved that the derived profiles will satisfy the same conditions as the original types.

In establishing practical profiles for various heights we can adopt either Type No. 1 or No. 2; but as the latter satisfies rigidly the four given conditions, and at the same time contains practically the minimum area consistent with these conditions, and with the necessity for simple outlines, it is to be preferred.

Although this type might be used until a pressure of about 14 tons per square foot were reached, we shall assume for our practical profiles a limiting pressure of 8 kilos. per square centimetre (8.19 tons of 2000 lbs. per square foot) at the front face, and 10 kilos. per square centimetre (10.24 tons of 2000 lbs. per square foot) at the back face, in order to keep within the limits usually recommended by engineers. The specific gravity of the masonry will be assumed as $2\frac{1}{3}$. From Practical Type No. 2 we obtain, by the simple method explained above:

Practical Profile No. 1 (Plate XVII., Table XX.)—Top width, 5 feet; height, 50 feet.
Practical Profile No. 2 (Plate XVIII., Table XXI.)—Top width, 10 feet; height, 100 feet.

The third practical profile which we shall give will be based upon Theoretical Profile No. 5 (see Plate X. and Table XII.). To make this profile a practical design we have only to simplify its outlines. Small changes in this respect will have very little influence upon the strength of the profile.

We have adopted a curved outline for the front face and a few straight lines for the back face, obtaining thus:

Practical Profile No. 3 (Plate XIX., Table XXII.)*—Top width, 18.74 feet; height, 200 feet.

This profile is the last illustration we shall give of the method we have advocated in this book, which consists in obtaining first a correct theoretical form, and then in simplifying its outlines. While we have required the theoretical profiles to fulfil rigidly the given conditions, it would evidently be a useless refinement to insist on the same accuracy for the practical design. As the theory of masonry dams has to be based upon hypotheses which are only approximately correct, we can permit the practical profiles to differ slightly from the given conditions.

How trifling the effect of changing the outlines of theoretical profile No. 5 has been will be seen by comparing Table XII. with Table XXII. Thus, the line of pressure P (reservoir full) remains within the centre third of the profile, and the line of pressure P' (reservoir empty) is only at two joints a little outside of this limit, viz.:

At a depth of 100 feet, 0.332 instead of 0.333
At a depth of 110 feet, 0.330 instead of 0.333

* The results given in this table have been checked by the graphic process, as explained in Note C, page 218 (see Plate XX.).

A greater eccentricity than this will be found in the types of Rankine and Harlacher (see page 34).

The maxima pressures at some of the joints are slightly in excess of the fixed limits, the greatest difference, however, amounting to less than 3 per cent.

In our Frontispiece we have compared Practical Profile No. 3 with the types designed by other engineers, and also with the profile proposed for the Quaker Bridge Dam.* As these types are based upon different data, a fairer comparison of their respective merits can be obtained by the following table, in which we have also included our Practical Profiles Nos. 1 and 2:

COMPARISON OF PROFILE-TYPES.

Type.	Top Width, in Metres.	Specific Gravity of Masonry.	Maxima Pressures, Kilos. per Sq. Centimetre.		Areas in Square Metres—Depths of Water.								
			Reservoir Full.	Reservoir Empty.	10 Metres.	15 Metres.	20 Metres.	25 Metres.	30 Metres.	35 Metres.	40 Metres.	45 Metres.	50 Metres.
De Sazilly	5.00	2.000	5.99	6.00	50.80	86.78	140.45	213.32	308.24	435.36	594.68	789.84	1027.89
Delocre..........	5.00	2.000	5.99	5.00	56.33	91.94	144.45	215.08	307.31	428.71	579.28	767.65	995.30
Rankine..........	5.71	2.000	7.55	9.80	70.34	118.48	176.97	249.08	337.56	446.02	579.20	742.68	943.54
Krantz..........	5.00	2.300	5.77	6.00	55.69	94.24	146.33	216.83	312.10	439.81	618.11	838.11	1099.81
Harlacher.........	4.00	2.200	6.73	5.75	44.35	79.85	132.95	205.75	300.40	418.92	...	....	....
Crugnola	4.75	2.300	7.72	8.27	51.29	87.84	140.73	214.02	309.07	427.27	578.27	761.77	996.11
Prac. Prof. No. 1..	1.53	2.333	3.49	3.58	33.52	74.77	...	....	....	....	....	....	....
" " No. 2..	3.05	2.333	6.98	7.16	37.62	76.78	134.05	208.25	299.15	....	....	....	....
" " No. 3..	5.71	2.333	8.20	10.09	59.52	96.86	149.51	218.55	305.68	412.31	541.64	696.12	878.39
Quaker Bridge Dam	6.10	2.500	9.08	10 44	64.24	102.20	151.21	219.42	298.88	402.74	531.83	687.98	871.20

The areas compared are entirely below the highest water-surface assumed for the respective types, the parts ot the profiles above the water-surface being omitted.

The maxima pressures given occur within a limit of 50 metres depth of water.

* See the Reports of B. S. Church, Chief Engineer, and of A. Fteley, Consulting Engineer, dated Julv 25th, 1887, published by the Aqueduct Commission.

CHAPTER VI.

CONSTRUCTION.

IN the preceding investigations we have considered only the profile of a dam. The next question which demands our attention is: How ought a reservoir wall to be built in plan, straight or curved? Both methods have been adopted successfully in practice, and the French engineers Delocre and Pelletreau have investigated this subject elaborately in their memoirs on reservoir walls. As there is, however, a great deal of uncertainty involved in the mathematics of curved dams, we shall enter only briefly upon this question.

When the valley which is to be closed by a reservoir wall is narrow, the idea naturally suggests itself to curve the plan of the dam, so as to make it form a horizontal arch, convex up-stream, transmitting thus the thrust of the water to the unyielding sides of the valley. That under such circumstances a wall may resist the water-pressure, when unable to do so merely by its weight, is proved by the Zola Dam (see page 53). We are led, therefore, to consider the two following questions:

1st. Under what circumstances will a curved dam resist as an arch?

2d. When it does act in this manner, can the profile be reduced from what would be required if the plan were straight?

With reference to the first of the above questions, it is known that a stone structure will not act as an arch if its thickness at the crown is too great with reference to the radius. The limiting value of this relation cannot be determined in the present state of our knowledge of the stability of arches, and it remains therefore a matter of judgment. M. Delocre thinks that a curved dam will act as an arch if its thickness does not exceed one third of the radius of its up-stream (convex) side. M. Pelletreau places the limiting value of the thickness at one half of this radius.

When a dam does act as an arch, it is evident that it can only transmit the water-pressure to the sides of the valley, and that its own weight must still be borne by the foundation. To investigate the horizontal thrust to which the masonry will be subjected under these circumstances, we will first imagine the dam to form part of a vertical shaft or well having to sustain the pressure of water only on its outer (convex) surface.

Such a structure ought evidently to have a circular plan, as it is subjected to a similar force all round.

Suppose the well to be divided into horizontal courses, each of them forming a ring composed of a number of voussoirs. As the only force acting on each ring in a horizontal direction is the water-pressure, it follows that the line of pressure (resistance) in each ring will form a circle passing through the centres of the voussoirs.

"The thrust round a circular ring under an uniform normal pressure is the product of the pressure on an unit of circumference by the radius."* It may therefore be expressed by the following formula:

$$T = pr, \tag{19}$$

* Professor Rankine's "Applied Mechanics," page 184.

in which

$T =$ the uniform thrust in the circular ring;
$p =$ the pressure per unit of length of the ring;
$r =$ the radius of the ring's outer surface.

Now if we remove part of the well and replace it by the practically rigid sides of the valley, we will have the case of a curved dam. The conditions in the masonry will remain unchanged, and the horizontal thrust in any course may be calculated by the above formula.

To find the maximum pressure exerted by this thrust on the masonry, we must know the position of the line of pressure. Pelletreau has assumed it to remain in the centre of each course, as in the case of a circular ring, the thrust being uniformly distributed on the masonry. Delocre places the position of the circular line of pressure on the up-stream limit of the centre longitudinal third of the course, and estimates, therefore, the maximum pressure on the masonry as twice the average pressure (formula B, page 10).

When a curved dam is subjected to the water-pressure it will yield slightly, owing to the elasticity of the masonry; but the sides of the valley will remain practically unchanged. It follows, therefore, that although we may calculate the horizontal thrust in any course by formula (19), as in the case of the circular well, the position of the line of pressure will be somewhat modified, approaching at the centre of the valley the up-stream face. The maximum pressure on the masonry will be greater than the average pressure, although probably not so large as assumed by M. Delocre.

It may be shown theoretically that, in the case of a narrow valley, a profile of less area may be adopted for a curved dam than for one whose plan is straight. M. Delocre comes to the conclusion that in either case, unless the height of the wall exceeds 84.85 metres (280.4 feet), its thickness at any given depth need never be greater than the width of the valley at that point. There is, however, so much uncertainty involved in the assumptions made in the mathematics of curved dams, that the best way to proceed in practice is to design the profile sufficiently strong to enable the wall to resist the water-pressure simply by its weight, and to curve the plan as an additional safeguard whenever the locality makes it advisable. This method is recommended by Rankine, Krantz, and other eminent engineers, and appears to have been generally adopted. Among the many curved dams described in this book, that of Zola (see page 53) is the only one which is unable to resist the water-pressure by its weight alone. But this reservoir wall is built under exceptional circumstances, its height being 120 feet and its length 205 feet. The maximum pressure on the masonry caused by this wall acting as a horizontal arch is only about eight tons per square foot.

It is evident that the advantage to be derived from curving the plan of a dam is confined to narrow valleys; for in the case of those of considerable width, requiring a large radius of curvature, the pressures in the masonry resulting from the dam acting as an arch are considerably in excess of what they would be if each section of the wall resisted simply by its weight. Should such a long dam not act as an arch, then the curving of the plan, by adding length to the wall, would involve a waste of material.

Having now investigated fully the proper manner of designing a masonry dam, both

as regards its profile and its plan, we will conclude our studies by considering how such a structure should be executed.

Unless the dam has but little height, its foundation ought unquestionably to be solid rock, into which the base and sides of the wall should be sunk a certain distance. All loose or soft portions of the rock must be removed, and its surface left as rough as possible. Such precautions will make it utterly impossible for the dam to slide on its foundation.

To prevent a loss of water from the reservoir, the rock should be stripped for a certain distance on the up-stream side of the dam, and all seams or fissures should be closed with mortar or masonry. The angle formed by the back face of the wall with the foundation ought to be rounded off with a heavy coating of mortar.

As regards the kind of masonry of which a reservoir wall may be built, we can select:

1. Cut stone masonry.
2. Rubble or concrete with cut-stone facings.
3. Concrete.
4. Rubble.

The first kind mentioned would seem at first thought to be the best for the purpose in view, on account of its great strength; but the following considerations make its use unadvisable:

1st. While offering only about twice the strength of rubble, it costs three to four times as much.

2d. As the form of the upper part of a dam depends upon the limitation placed upon the positions of the lines of pressure, and not upon the stresses in the masonry, the great strength of cut-stone work could only be utilized in the lower part of a high dam.

A combination of concrete or rubble with cut stone is not to be recommended. Owing to the difference in the settling of the two kinds of masonry, cracks and seams would be apt to result. This is seen in canal locks, where the cut-stone facing often becomes detached from the body of the wall.

Concrete is considered by many engineers to be too pervious a material to be placed in a dam. It has, however, been used successfully in the Geelong Dam (see page 81), and is to form the mass of a reservoir wall, 170 feet high, which is now being constructed near San Mateo for the San Francisco Water Works (see page 86).

Rubble masonry is undoubtedly the best material that can be used for building a dam. It possesses ample strength, adapts itself readily to any form of profile, and is comparatively cheap.

The great object to be attained is to form a monolith, as homogeneous as possible. Horizontal courses must therefore be avoided, and the stones interlocked in all directions. As regards the size of the stones to be employed, the mass of the wall may be composed of stones containing 2 to 7 cubic feet as in the Furens Dam (see page 53), or it may be formed of large blocks measuring 1 to 2 cubic yards as in the reservoir wall of Vyrnwy. The spaces between the large stones must be carefully filled with concrete or rubble made of small stones. Grouting ought never to be permitted, but efforts should be made

to form as dense a mass as possible. The best stones should be reserved for the facing, and should not be laid horizontal but normal to the faces. We have stated above that a masonry dam ought not to consist of courses. However, in building the wall, the masonry should be carried up uniformly in layers of about five feet thickness, the top of each layer being left as rough as possible, the projecting stones serving to bond it to the next course.

However carefully the masonry may be laid, a certain amount of leakage will always take place when the reservoir is first filled. This loss of water, which in a well-constructed dam shows itself only as a dampness on the front face, generally disappears soon. When the depth of the water in the reservoir is great, it is advisable to construct a "puddle-wall" against the back face for a certain height above the foundation, in order to insure the water-tightness of the wall. To relieve the base of the dam from the upward pressure of water leaking under it, a system of drains may be constructed in the foundation (see the Vyrnwy Dam, page 69) ; but such an arrangement is of doubtful utility, as it facilitates the leakage.

In connection with a masonry dam there are certain auxiliary constructions which must generally be provided, viz., an overflow-weir, outlet-pipes, and scouring-gates.

Unless a dam has been specially designed with a view of allowing water to flow over its crest, such a contingency should be rendered impossible by providing an ample overflow-weir. The shock of water falling on the front face of the wall will certainly injure the masonry, unless the profile has been designed to conform to the path described by the overflowing water. The weir may be constructed by keeping part of the dam at a lower elevation than the rest, giving it at the same time a suitable profile. A separate wall or tunnel may be used for this purpose. The size of the overflow-weir must be based upon the area of the watershed of the reservoir, the maximum rainfall, etc., and is to be determined by the usual hydraulic formulæ. As the whole safety of a dam may depend upon the correctness of this calculation, we must always aim to make the overflow ample beyond a doubt. Some formulæ for calculating the length of a waste-weir will be found on page 116.

The outlet-pipes may be imbedded in the masonry or placed in a gallery passing through the dam. If an open cut or tunnel has been excavated to divert the stream during construction, it is best to use it as a safe location for the outlet-pipes, the dam being left without any openings. The flow of the water from a reservoir is generally regulated by means of gates placed in a tower, which is often built as part of the dam (see the Alicante Dam, page 43).

When reservoirs are located in mountainous districts, where the streams, owing to the steep declivity of their beds, carry along considerable quantities of sand and gravel, they soon become filled with banks of deposited material, unless it is removed periodically. The best method of accomplishing this object is by means of a scouring-pipe or gallery. When this passage is opened the water forces its way through the deposited banks and washes most of the material out of the reservoir.

To illustrate the practical details of the construction of masonry dams and the accessory arrangements mentioned above, we have devoted the remaining pages of this book to short descriptions of the important reservoir walls of the present day. We refer the reader especially to the descriptions of the Furens, Vyrnwy, Titicus and New Croton dams.

CHAPTER VII.

SPANISH DAMS.*

The Almanza Dam [A] (Plate XXI.).—The oldest existing masonry dam is that of Almanza, situated in the Spanish province of Albacete, near the town after which it is named. The exact date of its construction is unknown, but it appears from old documents that it was in use prior to 1586.

It was founded on rock, and was built of rubble masonry, faced with cut stone except for the upper twenty feet of the front face, which was built of rough ashlar with courses of cut stones at certain intervals. The lower part of the dam, having a height of about 48 feet, is built on a curved plan, convex up-stream, the radius of the back face being 26.24 metres (86.07 feet). The remaining part of the wall, which was probably constructed at a later period, has a plan whose centre line forms a broken line 292 feet long. The greatest height of the dam is 20.69 metres (67.86 feet).

An overflow was formed by excavating the rock on one side of the dam 6.56 feet below its top for a length of about 39 feet.

Water is taken for the purpose of irrigation through a gallery 1 metre (3.28 feet) square, which passes through the lower portion of the wall. Above the down-stream end of this outlet-channel a chamber is constructed in the dam, where the bronze gate which regulates the flow of water from the reservoir is operated. To prevent the outlet-gallery from being closed by sediment, the gate is always partially opened during floods.

There is another gallery, 1.3 metres (4.26 feet) wide by 1.5 metres (4.92 feet) high, constructed through the dam, and serving for scouring out the deposits of sediment in the reservoir. In our description of the Alicante Dam we shall give a detailed account of the manner in which this operation is performed.

For many years the Almanza reservoir has not been filled, as the water is drawn off twice per annum for irrigation.

Although the old Spanish dam described above is not well proportioned, it is an interesting fact that its masonry has sustained safely for three centuries a greater pressure than exists in any other reservoir wall, namely, 14 kilos. per square centimetre (14.33 tons of 2000 lbs. per square foot).

The Alicante Dam [A] (Plate XXII.).—The highest Spanish dam is that of Alicante,—named also, after a village near its site, the Dam of Tibi. It was built during the years 1579 to 1594, to supply the arid region of Alicante with water for irrigation. Although the name of its constructor is not known with certainty, there are reasons for ascribing this work to Herreras, the famous architect of the Escurial palace.

The gorge of Tibi, which is closed by this dam, is formed entirely of hard calcareous rocks, the slopes on either side standing almost perpendicular. Its width is only 30 feet at the bottom, and 190 feet at the crown of the dam.

* The dams marked [A] are taken from "Irrigations du Midi de l'Espagne," par Maurice Aymard. Paris, 1864.

The river Monegre which flows through this gorge discharges on an average about 50 gallons per second.

The length of the Alicante reservoir is 5900 feet, its capacity being about 975,000,000 gallons.

The dam is built of rubble masonry faced with large cut stones. Its greatest height on the up-stream side is 41 metres (134.5 feet). The plan of the dam is curvilinear, the radius of the up-stream side of the crown being 107.13 metres (351.37 feet).

The maximum pressure in the masonry is 11.28 kilos. per square centimetre (11.54 tons of 2000 lbs. per square foot).

Water is taken from the reservoir by means of a well, 0.8 metre (2.62 feet) in diameter, which is placed in the dam itself. It is parallel with the up-stream face, and at a distance of about 2 feet from it. Fifty-one pairs of openings connect the well with the reservoir; they are 0.11 metre (0.36 foot) wide by 0.22 metre (0.72 foot) high, and are 0.3 metre (0.98 foot) apart horizontally and 0.41 metre (1.34 feet) vertically. The first pair is 6.97 metres (22.88 feet) below the top of the dam, and the last 2 metres (6.56 feet) above the bottom. By means of this arrangement water can be taken into the well even when much sediment has been deposited in the reservoir.

The outlet-well connects at the base of the dam with a horizontal gallery which is parallel with the up-stream face until it reaches the side of the gorge, where it is continued by a small tunnel 0.6 metre (1.97 feet) wide by 1.7 metres (5.58 feet) high. This tunnel curves so as to discharge the water parallel with the axis of the valley. The outlet-gallery is closed at the down-stream face of the dam by a bronze gate, two inches thick, giving an opening when completely raised of 1.77 feet width by 2.30 feet height. Immediately over the gate a small chamber has been cut in the rock, where the gearing for raising the gate is placed. By means of a hand-wheel and gear-wheels engaging a rack, which is attached to the gate, one man can raise or lower the latter with ease, even when the reservoir is full. To prevent incrustations which might obstruct the gate, a small stream of water is always allowed to escape there.

It was originally intended to have the outlet-gallery directly across the dam from face to face; but this would have brought it in close proximity to the scouring-gallery, presently to be described. Fears were entertained that such a construction would cause considerable leakage, and the outlet-gallery was therefore turned towards one side of the gorge, as described above.

We will now explain the construction and use of the scouring-gallery. Owing to the steep declivity of the beds of most Spanish streams, and to violent storms, large quantities of fine material which has been pulverized by the action of the water are deposited in the storage reservoirs. Unless some means were provided to remove this sediment, it would soon fill these basins completely. In 1843, when the Alicante reservoir had not been cleaned for fourteen years, a bank of sediment 75 feet high at the dam had been deposited. Since then the reservoir is scoured once in four years, the maximum height of the material deposited during that time being 39 to 52 feet.

Long experience has taught the Spaniards the best method of removing these deposits, namely, by means of scouring-galleries. In the Alicante Dam such a gallery is placed in the axis of the valley, crossing the dam in a straight line from face to face. Its

up-stream opening is 1.8 metres (5.9 feet) wide by 2.7 metres (8.86 feet) high. The gallery has this cross-section for the first 2.7 metres (8.86 feet) of its length, and is then suddenly enlarged to a section of 3 metres (9.84 feet) width by 3.3 metres (10.82 feet) height. After this the cross-section is increased gradually, so that it is 4 metres (13.12 feet) wide by 5.85 metres (19.18 feet) high at the down-stream face of the dam. By this increase in the cross-section of the gallery, which takes place in all directions, the material forced out of the reservoir by the water-pressure can expand freely and does not obstruct the channel through the dam.

The mouth of the scouring-gallery is closed simply by a timber bulkhead formed as follows: First a vertical row of beams about 1 foot square is placed, their ends projecting into horizontal grooves cut into the solid masonry. The last beam which closes the row is somewhat shorter than the rest, and enters only the lower groove. After the joints between the beams have been calked, a second row of similar timbers are placed directly behind the first row, but are laid horizontal, their ends being secured in vertical grooves in the sides of the gallery. Behind the second row three vertical posts are placed, each of which is firmly held by two inclined braces whose lower ends project into the floor of the gallery.

The banks of sediment formed in the reservoir acquire considerable consistency if left undisturbed for a few years. When it is necessary to scour the reservoir it becomes thus possible to remove gradually the timbers at the inlet of the gallery without much danger to the workmen. The timbers of the course next the reservoir are cut, one by one, with the greatest precaution. Should any movement be perceptible in the deposited material the men abandon their work, which will be quickly completed by the water-pressure.

Generally, however, the opposite of this takes place. The sediment forms a solid bank in front of the scouring-gallery, and does not move until a hole has been made through it from the top of the dam. The heavy iron bar which is employed for this purpose at the Alicante reservoir is 0.2 feet square, 59 feet long, and weighs about 1100 lbs. It is worked by means of a windlass and pulleys. When a hole has been pierced through the bank of sediment, the scouring action begins, first slowly, but soon gaining a tremendous force. All the sediment, except that in remote parts of the reservoir, is forced through the scouring-gallery, the noise made by this violent action being like that of cannons. Nothing remains for the workmen to do but to shovel the remaining sediment into the current. Sometimes the deposit has become so hard that it must be undermined from the scouring-gallery before a hole is pierced in it by the long bar. The total cost of scouring the reservoir, including the loss of timbers which are cut, amounts to only fifty dollars.

Although the method of cleaning the reservoir, which we have described in detail above, seems at first sight rather primitive, yet, on second thought, it will be found to be practical. Where such deep deposits are made gates are out of the question, as they would have to be frequently opened to prevent their becoming useless, and would cause thus a considerable loss of water.

While the scouring operation as carried on at the Alicante Dam certainly involves danger to the workmen, accidents are very rare. In our description of the Elche Dam we will show how this danger may be avoided.

The Alicante Dam had originally no waste-weir. However, one was built in 1697, as the wall was supposed to have been injured by water flowing over its top. During the freshets of 1792 the depth of the water on top of the dam was 8.2 feet, and it fell in a perfect cascade over the front face. The wall sustained this severe test so successfully that since then the waste-weir has been closed, no fears whatever being entertained of the stability and strength of the dam.

The cost of the construction of the great work we have described was borne entirely by the parties interested in the irrigation of Alicante.

The Elche Dam[A] (Plate XXIII.).—This reservoir wall is situated on the Rio Vinolapo, near the town of Elche. Like the dams already described it was founded on rock, and constructed of rubble masonry faced with cut stones. Its maximum height is 23.2 metres (76.1 feet).

The Elche reservoir is formed by three walls, which close converging valleys. The principal dam is about 230 feet long, measured on its crest, and is built according to a curved plan, the radius of the back face being 62.6 metres (205.38 feet).

No overflow-weir was provided, as perfect confidence was felt in the dam being able to withstand the flow of water over its crest without injury. In 1836, however, a considerable breach was made in the wall by water passing over it during a great flood.

In many details the Elche Dam resembles that of Alicante. Thus, water is taken from the reservoir by means of a vertical well which was built in the wall near its up-stream face, and has inlet openings at regular intervals. This well terminates in a horizontal gallery, which passes through the dam like that of Alicante, and has its down-stream end closed by a bronze gate operated from a chamber immediately above it.

The arrangement of the scouring-gallery, however, is a great improvement on that of the Alicante Dam. Immediately above it a working gallery is placed, which enables laborers to remove the last timbers of the gate which closes the scouring-gallery, with perfect safety. Above the scouring-gate there is a well-hole in the working gallery through which these timbers are pulled out by means of ropes.

The Puentes Dam[A] (Plate XXIV.).—The construction of the Puentes Dam was considered one of the great achievements of the reigns of Charles III. and Charles IV. of Spain. It was built during the years 1785 to 1791 at the place where the united waters of the Velez, Turrilla and Luchena form the Guadalantin River. After being in use for eleven years it was finally destroyed in 1802.

The maximum height of this dam was 50 metres (164 feet); its length measured on the crest was 282 metres (925.3 feet). The whole wall was built of rubble masonry, faced with large cut stones. The outlines of the plan were polygonal, being convex up-stream. The dam was finished with a magnificent parapet, upon which colossal statues of the two kings mentioned above were to have been placed.

According to M. Aymard, the maximum pressure in the masonry was 7.93 kilos. per square centimetre (8.12 tons of 2000 lbs. per square foot).

An arched scouring-gallery 6.7 metres wide by 7.53 metres high (22 feet by 24.7 feet) was constructed through the dam. At its up-stream end a central pier divided it into two channels, in order to reduce the span of the beams forming the scouring-gate.

Water was taken from the reservoir by means of two wells, each of which terminated

in a horizontal gallery 1.65 metres wide by 1.95 metres high (5.4 feet by 6.4 feet). These galleries were placed at different elevations, one being about 100 feet below the top of the dam, and the other near its base at the level of the scouring-gallery. The cross-section of the wells was about 4.2 metres by 2.5 metres (13.8 feet by 8.2 feet), and was rectangular, except that the side nearest the reservoir was formed by a circular arc. Each well had inlet openings, 0.28 metre wide by 0.55 metre high (0.92 foot by 1.80 feet), placed in rows of three, the vertical distance between the openings being 0.83 metre (2.72 feet).

According to the original intention the wall was to have been founded entirely on rock. In the centre of the valley, however, a deep pocket of earth was encountered, and it was unfortunately decided to build the wall at this place on a pile foundation. The masonry was sunk about 7 feet into the gravel around the piles, which projected above the horizontal caps. As the scouring-gallery and one of the outlet passages discharged in the centre of the valley, where the pocket of soft material was found, the ground at this place was protected against being washed out by a timber grillage resting on piles, which was continued for 131 feet down-stream from the front face of the dam. This timber apron was covered by about 7 feet of masonry, which was protected against the erosion of the water by planks.

The whole pile foundation was built very securely, and it would have answered all purposes had the depth of the water in the reservoir been less. This is shown by the fact that for eleven years, during which time the depth of the water in the reservoir never exceeded 82 feet, the wall stood perfectly safe. However, on the 30th of April, 1802, the water rose to an elevation of 154 feet above the base of the dam and the foundation gave way.

The following account of an eye-witness of the accident is taken from M. Aymard's book on the "Irrigation of the Southern Part of Spain":

"About half-past two on the afternoon of the 30th of April, 1802, it was noticed that on the down-stream side of the dam, towards the apron, water of an exceedingly red color was issuing in great quantities in bubbles, extending in the shape of a palm-tree. About three o'clock there was an explosion in the discharge-wells that were built in the dam from top to bottom, and at the same time the water escaping at the down-stream side increased in volume. In a short time a second explosion was heard, and, enveloped by an enormous mass of water, the piles and timbers which formed the pile-work of the foundation and of the apron were forced upwards.

"Immediately afterwards a new explosion occurred, and the two big gates that closed the scouring-gallery, and also the intermediate pier, fell in. At the same instant a mountain of water escaped in the form of an arc. It looked frightful, and had a red color, caused either by the mud with which it was charged, or by the reflection of the sun. The volume of water which escaped was so considerable that the reservoir was emptied in the space of one hour.

"The dam presents since its rupture the appearance of a bridge, whose abutments are the work still standing on the hillsides, and whose opening is about 56 feet broad by 108 feet high.*

* Mr. Crugnola states that this dam has been lately rebuilt ("Muri di Sostegno e Traverse dei Serbatoi," page 275.)

"At the moment of the accident the effective depth of the water was 33.4 metres (109.6 feet). Its surface was 46.80 metres (153.54 feet) above the base of the dam; the lower 13.40 metres (44 feet) being taken up by deposited material."

This fearful accident caused the loss of 608 lives, the destruction of 809 houses, and of property amounting to about 5,500,000 francs (1,045,000 dollars).

The cause of the failure of the Puentes Dam is seen clearly in the account we have given above. The wall was not overturned or crushed by the pressure it had to sustain, but failed because it was undermined by the great water-pressure forcing a way through the soft material in the centre of the valley.

The rupture of the Puentes Dam teaches the important fact that a high masonry dam, however well proportioned, will only be safe if founded entirely on rock.

The Dam del Gasco,[A] across the Guadarrama River, was commenced in 1788. Its general dimensions were to have been as follows:

	Metres.	Feet.
Height,	93	305.12
Thickness at base,	72	236.22
" " top,	4	13.12
Length on crown,	251	82.35

It was constructed on a straight plan, and was to consist of two walls, 2.8 metres (9.18 feet) thick, connected by cross-walls. The compartments which were thus formed were to have been filled with dry stones imbedded in clay.

In 1799, when the dam had already attained a height of 57 metres (187 feet), a heavy rain-storm caused the river to flow over its top. The swelling of the clay, resulting from its becoming wet, forced over part of the front wall, and the dam was never completed.

The Dam of the Val de Infierno[A] (Plate XXV.).—The region around the town of Lorca in the Spanish province of Murcia was formerly supplied with water for irrigation by the reservoir of the Val de Infierno. The masonry dam which forms this reservoir is situated in the gorge of the Rio Luchena, a branch of the Guadalantin River. Owing to the opposition of the land-owners below the site of the dam, who claimed that the scouring of the sediment injured their property, the reservoir has not been used for years, and is now completely filled with deposits. When the river is high it forms a beautiful waterfall over the old dam.

The greatest height of this reservoir wall is 35.5 metres (116.5 feet). It was originally intended to build the dam 16 feet higher, but this plan was abandoned as it would have caused the reservoir to include a permeable bank within its limits.

The plan of the dam has polygonal outlines, approaching very closely to arcs of circles, convex up-stream. The wall is founded entirely on rock.

An arched scouring-gallery, having a uniform height of 4.5 metres (14.8 feet) and a width of 3.75 metres (12.3 feet), except for 16.4 feet from its up-stream end, where its width is only 2.75 metres (9.0 feet), passes through the wall.

There are also two outlet-galleries, placed at different levels and arranged like those

of the dams of Alicante and Elche. The vertical wells with which they are connected at the up-stream face of the dam have inlet-openings 0.3 metre wide by 0.5 metre high (0.98 foot by 1.64 feet), placed 3 metres (9.84 feet) apart. This distance is too great. When the deposits close the openings at one level, no water can be drawn out of the reservoir until the water-surface has been raised about ten feet. Part of the up-stream sides of the wells has been torn down in order to facilitate the drawing of water, which makes the dam practically like the primitive one of Almanza, where no wells at all were used.

The reservoir of the Val de Infierno was constructed during the years 1785 to 1791.

The Nijar Dam[A] (Plate XXVI.).—This reservoir wall is situated in a gorge of the Carrizal River in the small village of Nijar, near the town of Almeria. It was designed by the architect Geronimo Ros, and was constructed during the years 1843 to 1850. This dam was founded on rock and built of rubble masonry faced with cut stone. The lower portion of the wall consists of a foundation-mass of masonry, having a width in the direction of the valley of 43.89 metres (144 feet). This masonry extends 11 metres (36.1 feet) down-stream and 12.29 metres (40.3 feet) up-stream beyond the wall proper. The down-stream face of this foundation-mass is carried up in steps, as shown in Plate XXVI. The maximum height of the dam above the bed of the river is 30.93 metres (101.5 feet).

A scouring-gallery, 1 metre wide by 2.19 metres high (3.3 feet by 7.2 feet) passes through the wall. At its up-stream entrance it is only 1.72 metres (5.6 feet) high. It is closed by a gate which is operated from the top of the dam by means of a long rod. Immediately over the gate there is a vertical well, 1 metre in diameter, in the wall, which enables the workmen to examine the gate without being exposed to danger.

Water is drawn from the reservoir by means of a vertical well and a horizontal gallery, as in the other dams we have described. The diameter of the well is 2.72 metres (8.9 feet). A winding staircase in the well affords opportunity for closing up the inlet-holes, when necessary for repairs.

The overflow-weir consists of two openings, 2.2 metres wide by 1.6 metres high (7.2 feet by 5.2 feet), whose sides are placed 1.6 metres (5.25 feet) below the top of the dam.

The capacity of the reservoir is about 5,475,000,000 gallons, but the water-surface is never above half the height of the dam.

The Lozoya Dam[A] (Plate XXVII.).—About the middle of this century the engineers of the Spanish government constructed a canal, known as that of Isabella II., for supplying Madrid with water from the Rio Lozoya. As the natural surface of this river is not sufficiently elevated for this purpose, it was raised by means of a masonry dam 32 metres (105 feet) high, and 72.5 metres (237.8 feet) long on top.

This dam consists of a wall of cut stone, 18.66 metres (61.2 feet) thick at the base, backed by rubble masonry, making the total thickness of the dam at its base 39 metres (128 feet). The back face is partially covered by a slope of gravel. The plan of the dam is straight.

No galleries pass through the wall; they are driven through the rocky banks of the reservoir. On the right bank there are two galleries; one, 6.82 metres (22.4 feet) below the

crown of the wall, serves to feed the canal, and the other, placed 9.1 metres (29.86 feet) below the same level, is the scouring-gallery, below which the reservoir is allowed to fill up with deposits.

On the left of the dam there is an overflow-weir cut in the rock, 8.4 metres (27.6 feet) wide, and 3.35 metres (11 feet) below the top of the wall.

The Villar Dam* (Plate XXVIII.).—The Villar reservoir on the river Lozoya was constructed in 1870–1878, to furnish an additional supply of water for Madrid. Mr. José Morer, chief engineer to the Spanish government, designed the reservoir and dam. The work was commenced in 1870 and completed in 1878.

The dam is about 170 feet high, and forms a reservoir having a capacity of about 4,400,000,000 gallons. It was built on a curved plan, the radius being 440 feet. The length on top is 546 feet, of which 197 feet are 8′ 3″ lower than the rest in order to form an overflow-weir. The maximum depth of the water below the level of the overflow-weir is 162 feet. Four lateral tunnels serve to discharge the excess of water in case of floods.

Two galleries run through the dam at a depth of 143 feet below the level of the overflow. Each gallery has an inlet of nineteen square feet, divided into two compartments, which are closed by sluices. These are operated by means of hydraulic power from a central tower, which is built on the inner side of the dam up to the level of the roadway.

With the exception of some cut stone on the crown, the whole dam was built of rubble masonry.

The total cost of the reservoir and dam was about $390,000.

The two Hijar dams† (Plate XXIX.) were built in 1880 on the Martin River, at a distance of about nineteen miles from the city of Hijar, in order to form two large reservoirs for irrigation purposes. The first has a capacity of 6,000,000 cubic metres (1,584,846,000 gallons), supplied from a watershed of 238 square kilometres (92 square miles), and the capacity of the second is 11,000,000 cubic metres (2,905,551,000 gallons), its watershed containing 43 square kilometres (17 square miles).

Each reservoir has a masonry dam whose general dimensions are:

	Metres.	Feet.
Length on top,	72	236.22
Height,	43	141.07
Width at top,	5	16.40
" " 9 metres below top,	5.2	17.06
" " base,	44.8	146.98
	Square Metres.	Square Feet.
Area of profile,	785.45	8,453.8

The back face of the profile is formed of a vertical line to a depth of 25 metres (82.02 feet), from which point it is continued by a circular curve, whose versed-sine at the base is 6.50 metres (21.33 feet). The front face is formed of a sloping line to a depth of 9 metres

* Proc. Inst. C. E., vol. 71, page 379.

† "Bacini d'irrigazione," per G. Torricelli.

(29.53 feet), and then of a series of steps 2 metres (6.56 feet) high, and having an average width of 1.50 metres (4.92 feet). The outer corners of these steps are located in a circular arc, concave down-stream.

The maximum pressures on the masonry are:

	Kilos. per sq. centimetre.	Tons of 2000 lbs. per sq. foot.
Reservoir full, ...	5	5.12
Reservoir empty, ...	5.86	5.99

Both of the Hijar dams are founded on rock, and are built circular in plan, the radius being 64 metres (210 feet).

CHAPTER VIII.

FRENCH DAMS.*

THE masonry dams built in France prior to the publication of M. de Sazilly's "Profile of Equal Resistance" in the "Annales des Ponts et Chaussées" for 1853 have less extravagant profiles than the old Spanish dams, but show, nevertheless, an utter absence of any rational theory regarding the proper method of designing a masonry dam. The difference of opinion held formerly by engineers as to which side of the profile ought to have the greater horizontal projection is shown in the following profiles, which we have taken from M. Krantz's "Study on Reservoir Walls:"†

Lampy Dam (Plate XXX.), built in 1776–1782, on the canal of the South.
Vioreau " (Plate XXXI.), " " 1833–1838, on the canal from Nantes to Brest.
Bosmelea " (Plate XXXII.), " " 1833–1838.
Glomel " (Plate XXXIII.), " " 1833–1838, on the canal from Nantes to Brest.

The Gros-Bois Dam T (Plate XXXIV.) was constructed in 1830–1838 on the Brenne River to form a reservoir for feeding the canal of Bourgogne. Its principal dimensions are:

	Metres.	Feet.
Length on top,	550.00	1804.6
Height above river-bed,	22.30	73.2
" " foundation,	28.30	92.9
Width at top,	6.50	21.32
" " base,	14.00	45.9

The overflow-weir is 10 metres (32.81 feet) long and 3 metres (9.84 feet) below the top of the wall.

The foundation upon which this dam was built consists of argillaceous rock possessing little hardness. When the wall had attained a height of only 4 metres (13.12 feet), a serious leak occurred through the foundation. Some lime was thrown near the crevices produced by the leak, but did not stop the loss of water. The reservoir had, therefore, to be emptied, and the crevices closed with masonry. In 1837 the tunnel which had been used as a waste-weir during the construction was closed, and the water allowed to fill the reservoir. When it had reached a depth of 17.45 metres (57.25 feet) its pressure produced a fissure at the intersection of the dam with the tower of the gate-house. It was noticed that the wall deflected a few centimetres down-stream under this pressure, and, upon the reservoir being emptied, returned almost to its original position. This fact proves that masonry has considerable elasticity.

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* The dams marked T are taken from "Bacini d'Irrigazione," per G. Torricelli. Roma, 1885.

† Paris, 1870.

In addition to this deflection, it was soon noticed that the wall had slid 0.045 metre (0.15 foot) down-stream. To arrest this motion the dam was reinforced in 1842 by seven counterforts, each being 4 metres (13.12 feet) thick on top and 11.30 metres (37.08 feet) at the base, projecting 8 metres (26.25 feet) from the front face. As fissures, however, were still noticed in the foundation, two more counterforts were built, and stopped all further trouble.

The Tillot Dam T is 13 metres (42.65 feet) high above the bed of the river, and 20 metres (65.62 feet) from the foundation. It has both faces vertical, and is 5.45 metres (17.88 feet) thick.

The Chazilly Dam T is situated in the Sabine Valley near Chazilly. It is 22.50 metres (73.80 feet) high, and 536 metres (1758.62 feet) long on top. Its thickness is 4.08 metres (13.39 feet) on top, and 16.20 metres (53.15 feet) at the base. It was built according to the profile of the Gros-Bois Dam (see Plate XXXIV.).

The Zola Dam (Plate XXXV.) was built about the year 1843 to form a reservoir for supplying the city of Aix (Provence) with water. It is named after M. Zola, the engineer who projected its construction but died before the plans were matured. The general dimensions of this dam are as follows:

	Metres.	Feet.
Length on top,	62.5	205.00
" at base,	7.0	22.96
Height above foundation,	36.5	119.76
Width at top,	5.8	19.02
" " base,	12.75	41.82
	Square Metres.	Square Feet.
Cross-section of wall,	338.62	3644.6

The wall is surmounted by a parapet 1.20 metres (3.94 feet) high.

The Zola Dam is built of rubble masonry and made circular in plan, the radius at the crown being 158 feet. This dam is the only one known to the writer which is unable to resist the thrust of the water by its weight alone, and owes, therefore, its stability solely to its acting as a horizontal arch abutting against the sides of the valley. Assuming the specific gravity of the masonry as 2.2, we find that when the reservoir is full the resultant pressure at the base lies 3.5 metres (11.48 feet) outside the wall. At 9 metres (29.52 feet) height it would be 2.50 metres (8.20 feet) outside of the front face. At a height of 19 metres (62.32 feet) the resultant would be 2.75 metres (9.02 feet) inside of the wall, causing a maximum pressure on the masonry of 7.93 kilos. per square centimetre (8.12 tons of 2000 lbs. per square foot).

We have taken the above description from the memoir on the Verdon Dam by M. Tournadre, published in the "Annales des Ponts et Chaussées" for 1872 (1st semestre).

The Furens Dam (Plate XXXVI.).—This dam is also known as that of the "Gouffre d'Enfer," the name of the gorge which it closes; also as the dam of Rochetaillée, the name of a village near its site; and as the dam of Saint-Etienne.

In 1858 the French government decided to construct an immense reservoir in the valley of the Furens River in order to protect the town of Saint-Etienne from inundations.

The total cost of the work was estimated at $298,300, of which amount the town of Saint-Etienne agreed to pay $190,000 for the privilege of using part of the reservoir for storing water.

The mean annual flow of the Furens River is about 130 gallons per second, but in dry seasons it amounts to only 21 to 26 gallons per second. In 1849 the town of Saint-Etienne was inundated, owing to a great rise of the Furens River, caused by the bursting of a water-spout. According to the calculations of the French engineers the discharge of the Furens at that time must have amounted to about 34,600 gallons per second. The reservoir was designed to prevent inundations even in case of a similar maximum discharge. The drainage area of the Furens above the reservoir site is 9.65 square miles, and the mean annual rainfall 39.4 inches.

The engineers who designed and constructed the Furens Dam and reservoir are: M. Græff, the Chief Engineer of the Département of the Loire; M. Delocre, who made the theoretical studies of the best form of profile; and M. Montgolfier, who had charge of the construction. M. Conte-Grandchamps assisted in the preliminary studies, but was promoted to another position before the masonry was commenced.

The greatest depth of water in the reservoir is 50 metres (164 feet), the total storage capacity being 1,600,000 cubic metres (422,625,000 gallons). Of this, however, the town of Saint-Etienne is only allowed to utilize 1,200,000 cubic metres (316,969,000 gallons), corresponding to a depth of water of 44.5 metres at the dam. The remaining 400,000 cubic metres (105,656,000) gallons of storage are reserved for preventing inundations.

The outlet from the reservoir consists of two cast-iron pipes, 0.40 metre (1.31 feet) in diameter, which pass through a lateral tunnel.

In constructing a dam exceeding in height all that were then existing, it is not astonishing that the engineers in charge of the work adopted out of precaution the low limit of pressure of 6½ kilos. per square centimetre (6.64 tons of 2000 lbs. per square foot), although they knew that some of the old Spanish dams sustain much greater stresses.

As the gorge which was to be closed was very narrow, it was decided to make the plan of the dam curvilinear, the radius being 252.50 metres (828.38 feet). The chord at the crown of the wall is 100 metres (328.07 feet), having a versed-sine of 5 metres (16.4 feet). This is the first French dam that was built curvilinear in plan.

The profile was based upon the type proposed by M. Delocre, which is shown in Plate II.; but curvilinear outlines were adopted in order to produce a more pleasing appearance. The thickness at the top of the dam was increased on account of the danger from floating masses of ice; but at the bottom the width of the profile is slightly less than in Delocre's type.

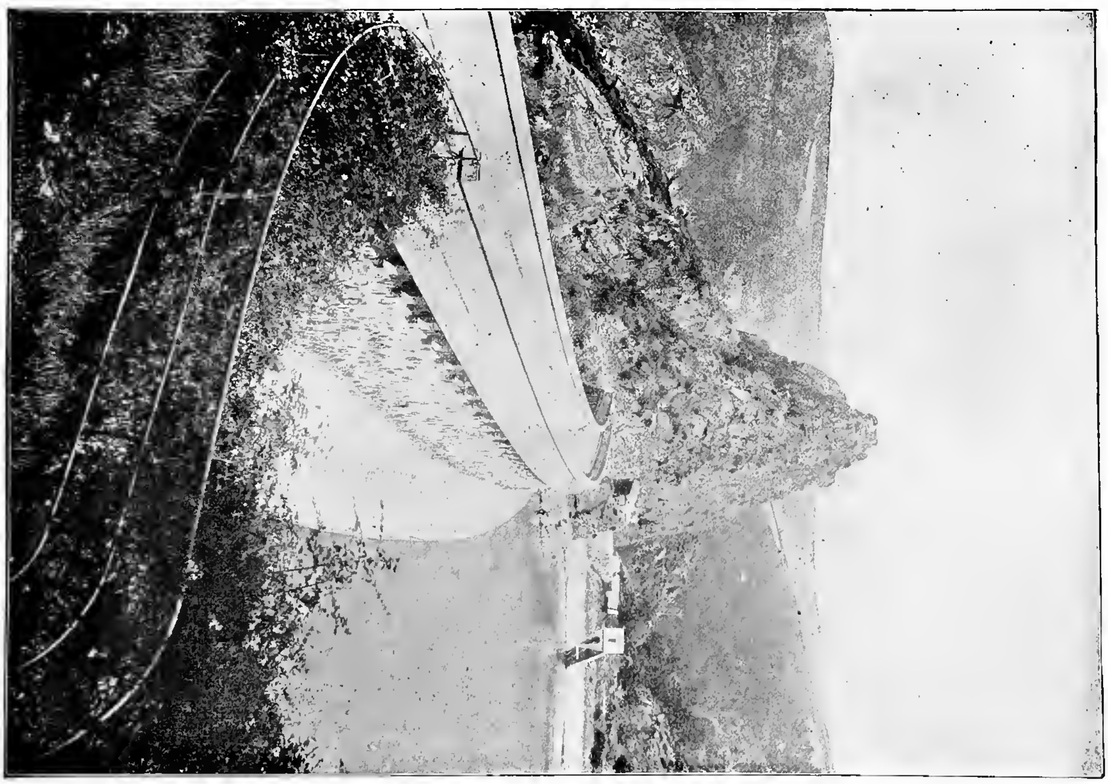
The greatest height of the dam above the foundation is 56 metres (183.72 feet) on the down-stream side, but up-stream it is only 52 metres (170.6 feet).

Great pains were taken in all the details of construction. Before excavating the foundations a new, permanent channel was made for the Furens, and two lateral tunnels were also excavated to serve subsequently for the outlet-pipes. By these means, and a coffer-dam, the foundation was kept perfectly dry.

The rock on which the dam was built was mica schist. All loose or seamy portions were removed, and the whole foundation was sunk at least one metre into the rock, in

PLATE A.

FURENS DAM. (Front.)



FURENS DAM. (Back.)

order to prevent the dam from sliding. Where the surface of the rock was smooth, it was roughened either by exploding petards or else by coating it with Vassy cement into which building-stones were stuck.

The whole wall, including the facing, was built of rubble masonry, except the angle of the upper retreat, the parapets, and the corbels upon the outside facing. The stones were procured from the excavation for the foundation, from the new channel for the river, and from two neighboring quarries. The best stones were selected for the faces, where they showed a section of about 1×1.6 feet and joints of $\frac{3}{4}$ to $1\frac{1}{4}$ inches. The stones varied in size from 2 to 7 cubic feet. In order to prevent unequal settling, the masonry was carried up about 5 feet high at a time over the whole wall. The top of each of these layers was left with as many projecting stones as possible, so as to bond it firmly with the next layer.

Cut stones 2.6 feet long and about 1.1 feet high were placed in the front face in quincunx order, 4.6 metres (15.09 feet) from centre to centre, and projecting 1.3 feet. On the back face there are three rows of iron rings to facilitate repairs.

The thickness of the dam at the highest level where the water is stored—44.5 metres (146 feet) above the bottom of the reservoir at the dam—is 6.37 metres (20.9 feet). Above this there is a guard-wall 5 metres (16 feet) high, having a thickness of 3.75 metres (12.3 feet) at the base and 3 metres (9.84 feet) at the top. This furnishes room for a carriage-way and two foot-paths. On the top of the guard-wall there are two parapets which add 0.5 metre (1.64 feet) to the total height of the dam.

In order to prevent all leakage from the reservoir, the rock was stripped on the up-stream side of the dam for 60 to 80 feet, and all fissures that were discovered were carefully sealed with cement or masonry. Great pains were taken to make the joints between the dam and the rock on each side perfectly water-tight. A coating of 3 to 4 inches of cement was placed at the angles formed by the facing of the dam with the rock into which the dam was imbedded. It was originally intended to make all the joints of the up-stream face with Vassy cement, but this plan was discontinued after the dam had been built about 49 feet high, as the introduction of water into the reservoir proved that ordinary mortar would answer just as well.

Owing to the high altitude of the Furens Dam, work on the reservoir could only be carried on from May 1st to October 1st of each year. The balance of the time, however, stones were quarried and hauled to convenient positions. The materials required in construction were distributed by means of a railway located on the top of the wall, and which was raised as the work advanced.

The masonry of the dam was commenced in 1862 and completed in 1866. Four superintendents and twenty-five to thirty masons were employed, and laid on an average $10\frac{1}{2}$ yards of masonry per day. The actual number of working days did not exceed 120 per annum. The total quantity of masonry in the dam was 52,300 cubic yards. The cost of impounding the water was 1.15 francs per cubic metre (0.0062 dollar per cubic foot).

The work of each season was allowed to harden thoroughly, and was then tested by allowing the water to flow into the reservoir and finally over the dam. In December, 1865, there were 46 metres (150.91 feet) of water in the reservoir; in March, 1866, 47 metres (154.19 feet). The only effect produced by this great water-pressure was a dampness on

the front face, which was doubtless due to the porosity of the stone and mortar. A ditch was dug in the front of the dam and left open for four months in order to detect any leakage, but remained perfectly dry.

The description we have given above of the construction of the Furens Dam has been taken from the interesting memoirs published in the "Annales des Ponts et Chaussées" by MM. Græff and Delocre in 1866, and by M. Montgolfier in 1875.

In concluding the brief account we have given of this great work, we will state that the Furens Dam is the highest reservoir wall of the present time, and that it is the first masonry dam, which has been built in accordance with correct scientific principles. Its construction has been in every respect a great success, owing to the care taken in all the details of the building by the eminent engineers who directed the work.

The Pas du Riot Dam[T] is situated about 8200 feet from the Furens Dam, and was constructed in 1872–78 in order to form a reservoir of 1,300,000 cubic metres (343,383,000 gallons) capacity for the city of Saint-Etienne. The dam is 34.50 metres (113.19 feet) high, and is built on a curve in plan. Its profile was based upon that of the Furens Dam.

The Cotatay Dam[T] was built in 1885 on the Cotatay Brook, near Saint-Etienne, to supply the city of Chambon-Fengerolles with water. The dam is 34.50 metres (113.19 feet) high. Its profile was based upon that of the Furens Dam.

The capacity of the reservoir is 2,000,000 cubic metres (528,282,000 gallons).

The Vingeanne Dam[T] (Plate XXXVIA) was constructed in 1885 near the town of Baissey. Its height is 34.70 metres (113.85 feet); its width is 3.5 metres (11.48 feet) at top and 24.42 metres (80.10 feet) at the base. The profile was designed by taking the obliquity of the resultant pressure, reservoir full, into account in accordance with the method of M. Guillemain (page 13).

The Ternay Dam (Plate XXXVII.).—This dam was built to prevent inundations by the Ternay River, and to supply the town of Annonay, in the province of the Ardèche, with water. The costs of the construction were borne by the state, town, and manufacturing interests.

The account we give of this work is taken from the interesting memoir published in the "Annales des Ponts et Chaussées" for 1875 by M. Bouvier, who designed and constructed the dam and reservoir under the general directions of M. Krantz, the Chief Engineer.

The vertical pressures in the masonry were not to exceed 7 kilos. per square centimetre (7.16 tons of 2000 lbs. per square foot); the coefficient of friction necessary to prevent sliding was limited to 0.76. The profile of the dam proper is surmounted by a rectangular portion forming a guard-wall 3.65 metres (11.97 feet) high and 4 metres (13.12 feet) thick. The dam without the guard-wall is 34.35 metres (112.67 feet) high and 4.8 metres (15.74 feet) thick on top.

The up-stream side of the profile is formed first by a vertical line 17.85 metres (58.56 feet) long, and then by two inclined lines, the first having a slope of 0.8 metre (2.62 feet) horizontal to 5.5 metres (18.04 feet) vertical; the second a slope of 3 metres (9.84 feet) to 11 metres (36.09 feet).

The down-stream side of the profile is formed by a circular curve whose radius is 45 metres (147.63 feet). The centre of the circle is 2.3 metres (7.54 feet) above the

crown of the dam. The curve terminates 9.3 metres (30.51 feet) above the base of the dam, the front face being completed by a tangent to the curve, whose horizontal projection is 7.1 metres (23.29 feet). The total thickness of the dam at its base is 27.2 metres (89.25 feet).

In his memoir describing the Ternay Dam, M. Bouvier advanced the formulæ we have given on page 12. The maximum pressure in this reservoir wall calculated by these formulæ amounts to 9 kilos. per square centimetre. M. Bouvier cites the experiments of M. Vicat to show that good hydraulic mortar may safely sustain pressures of 14.4 kilos. per square centimetre (14.73 tons of 2000 lbs. per square foot).

The Ternay Dam was built of rubble masonry, the stones used being granite. M. Bouvier gives:

The specific gravity of the granite,	2.620
" " " " " mortar,	1.970
" " " " " masonry,	2.35

The proportion of the stone to mortar was as 6 to 4.

The wall was built on a curved plan, the radius being 400 metres (1312 feet).

The capacity of the reservoir formed by the Ternay Dam is 2,600,000 cubic metres (686,-766,000 gallons), which is sufficient storage-room for retaining all the flood-waters of the river.

The dam and reservoir were built in 1865–1868.

The Ban Dam[T] (Plate XXXVIII.) was constructed in 1867–1870 to form a reservoir of 1,800,000 cubic metres (475,454,000 gallons) capacity, from which the city of St. Chamond draws its water-supply. This dam is 46.30 metres (151.86 feet) high. It was built on a curved plan. The profile was determined by the method employed for the Furens dam, but a higher limit of pressure was taken, namely, 8 kilos. per square centimetre (8.18 tons of 2000 lbs. per square foot). The dam is founded on rock, and composed of rubble masonry.

The Verdon Dam (Plate XXXIX.) was constructed in 1866–1870 to raise the level of the Verdon River sufficiently high to feed a canal which supplies the city of Aix (Provence) and other places with water. At the site selected for this work, near the village of Quinson, the width of the valley is only 115 to 130 feet, its rocky sides being almost vertical and about 160–200 feet high. The Verdon River has a fall of .003 foot in 1 foot, and its flow varies from 10 to 1200 cubic metres (2640–317,000 gallons) per second. To construct a dam across a river subject to great freshets, founding the wall on solid rock after excavating about 20 feet of gravel and boulders, was a difficult undertaking. For a detailed account of how the work was executed we must refer the reader to the interesting memoirs published by the Chief Engineer, M. de Tournadre, in the "Annales des Ponts et Chaussées" for 1872 (1st semestre), from which we take the following description. The general dimensions of the dam are:

	Metres.	Feet.
Length,	40.00	131.23
Height above river-bed,	12.25	40.19
" " foundation,	18.00	59.06
Width at top,	4.32	14.17
" " base,	9.91	32.51
" of foundation,	15.00	49.21

The design of the dam was based on the assumption that it would be submerged 5 metres (16.4 feet) when the river had its maximum flow of 1200 cubic metres (317,000 gallons) per second. The profile had, therefore, to be made very strong, and special precautions taken in the details of the construction. A great freshet occurring soon after the completion of the work proved the correctness of the above assumption.

The foundation is built of concrete, and the dam proper of rubble with a cut-stone facing down-stream and a heavy cut-stone coping 0.75 metre (2.46 feet) thick. This coping forms six courses of voussoirs in plan, the stones being fastened together by iron clamps, and also secured to the up-stream side by iron dowels.

The mortar used in the masonry was composed of hydraulic lime of Theil and river sand.

The Verdom Dam is built circular in plan, the radius being 33.171 metres (108.83 feet) for the front face at the base. The foundation-mass of concrete, however, has a rectangular plan.

To resist the high fall of the water passing over the dam during freshets, a rip-rap of large boulders is placed in front of the wall.

Bouzey Dam* (Plate XL.) was constructed in 1878 to 1881 near Epinal, France, to form a reservoir of about 1,875,000,000 U. S. gallons capacity for the "Canal de l'Est." The dam had a length of about 1700 feet on top. Its greatest height was 49 feet above the river-bed and 72 feet above the foundation. The dam was built straight in plan and had a profile which was calculated to be one of "equal resistance," each joint being assumed perpendicular to the resultant of all the forces acting on it. The profile adopted did not originally include the shaded portions shown in Plate XL.

The dam was founded on red sandstone, which was fissured and quite permeable. Considerable difficulty was experienced in the foundation-trench from springs. To prevent leakage under the dam a guard-wall, two metres thick, was built at the up-stream face from the solid rock to the river-bed, but the foundation of the dam itself was only excavated to fairly good bottom and not to the solid rock.

The dam was completed in 1880, but the reservoir was not filled until about a year later. When the water reached a level 33 feet below the top of the dam, springs of about 2 cubic feet per second appeared on the lower side of the wall. This leakage was partly due to two vertical fissures which had been made in the wall by changes of temperature before the reservoir was filled.

The water was raised very gradually in the reservoir. When it reached, on March 14, 1884, a level 10.5 feet below the top of the dam, a portion of the wall 444 feet long was shoved forward so as to form a curve, convex down-stream, having a versed sine of 1.1 feet. Four additional fissures appeared at the same time in the front face of the dam and increased the flow of the springs in front of the wall to about 8 cubic feet per second. No further motion took place in the dam although the water was kept at the level it had reached. The fissures in the wall opened in the winter and closed in summer on account of changes of temperature, their average width being about 0.28 inch.

* Le Génie Civil for 1895 and Proc. Inst. C. E., vol. cxxv.

In 1885 the water was allowed to rise to the high-water level (1.97 feet below the top of the wall), and the reservoir was then emptied for inspection. It was found that the dam had separated from the guard-wall for a stretch of about 97 feet when it had been shoved forwards, and many fissures were discovered on the inner face. The masonry was repaired. To prevent the dam from sliding, an abutment was built in front of it and connected by an inclined wall which was toothed into the dam. A block of masonry was, also, built on the up-stream face to close the joint opened between the main dam and the guard-wall, and was surrounded by a bank of puddle, about 10 feet thick. The masonry added to the dam is shown by the shaded portions in Plate XL. Drains were placed in the masonry to carry off any water that might leak under the dam. The repairs mentioned were begun in 1888 and completed by September 14, 1889. The water was admitted to the reservoir again in November, 1889. On April 27, 1895, the water being at its highest level, about 594 feet of the central part of the dam was suddenly overturned at a plane about 33 feet below the top of the wall. The fracture was almost horizontal longitudinally. It was level transversely for about 12 feet and then dipped toward the outer face. The accident caused a great loss of life and property.

The failure of the Bouzey Dam is supposed to have been due to a greater tension at the up-stream face than the masonry could resist. This tension was probably increased by an upward water-pressure under the dam, which had not been founded on an impervious stratum.

The Pont Dam[*] (Plate XLI.) was built in 1883 on the Armaçon River, at a distance of 2½ miles from the city of Sémur. The dam is circular in plan, having a radius of 400 metres (1312.40 feet) and a versed-sine of 7.10 metres (23.30 feet). The length of the dam on top is 150.89 metres (495.12 feet).

The profile has a top width of 5 metres (16.4 feet), and is bounded as follows: on the back by a straight line having a batter of 0.05 metre per 1 metre of height; on the front, by a circular arc whose radius is 30 metres (98.43 feet) to a depth of 19 metres (62.34 feet) from the top, and which is continued by a tangent.

The height of the dam proper is 20 metres (65.62 feet), to which the foundation adds about 6 metres (19.69 feet). There are 7 counterforts on the front face, 5 metres (16.40 feet) wide by 3 metres (9.84 feet) thick, inclined parallel to the front face of the dam.

The dam was founded on rock and built of granite. When the reservoir was first filled, some water leaked through the dam, but the filtrations soon disappeared and the wall is now in excellent condition.

The Chartrain or Tâche Dam[*] (Plate XLI. *A*) was constructed in 1888–92 on the Tâche, an affluent of the river Renaison, which flows into the Loire to form a reservoir for supplying the city of Roanne with water.

The capacity of the reservoir is 4,500,000 cubic metres (158,897,000 cubic feet), of which, however, 500,000 cubic metres have to be reserved for storm-water.

The reservoir has a surface of 22 hectares (54.36 acres) and is supplied from a water-shed of 1400 hectares (52 square miles).

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* Vth International Congress on Inland Navigation. Report by M. Marius Bouvier on the Reservoirs in the South of France.

The Chartrain Dam is the most recent example of the construction of a dam in France built according to a scientific profile. Its design was based on the following principles:

1st. The lines of pressure, reservoir full or empty, must be kept within the centre third of the profile.

2d. The maxima pressures in the masonry or on the foundations are not to exceed 11 kilogrammes per square centimetre.

3d. There must be no possibility of the dam's sliding or shearing apart.

The plan of the dam was curved to a radius of 400 metres.

The Chartrain Dam was constructed of rubble masonry, made of porphyric rock and hydraulic mortar, weighing about 2400 kilogrammes per cubic metre (150 lbs. per cubic foot). The up-stream face was covered with a layer of artificial cement of slaked lime, 0.03 metre thick, made of equal parts of cement and sand, to 10 metres below the coping. In spite of this coating there was considerable leakage through the dam at first. It is, however, steadily diminishing.

The total cost of the Chartrain Reservoir was $2,100,000 or 0.47 franc per cubic metre stored.

The Mouche Dam * (Plate XLII.), completed in 1890, was constructed across the Mouche River, an affluent of the Marne, near the village of Saint-Ciergues, to form a storage reservoir of 8,648,000 cubic metres (305,365,000 cubic feet) capacity for storing water for "the canal of the Haute-Marne." The surface of the reservoir at the level at which the water is to be stored is 97 hectares 46 ares (241.83 acres).

The Mouche Dam was designed by M. Carlier, Chief Engineer. It is 410.25 metres (1346 feet) long and is built straight in plan. The depth of the water in the reservoir above the meadow of the thalweg is 28.98 metres (95.08 feet).

As no good material for an earthen dam could be found near the site selected, it was decided to construct a masonry reservoir wall, although this involved an excavation of 7–12 metres below the surface of the ground to reach the marl-rock on which the dam was founded. 56 per cent of the total masonry of the dam was laid below the surface of the ground.

The profile of the dam (Plate XLII.) was determined by the method recommended by M. Bouvier and improved by M. Guillemain. Besides fixing a limit for the pressure to be permitted in the masonry, the French engineers adopted also the condition, first insisted upon by Prof. Rankine, that the lines of pressure, reservoir full or empty, should be kept within the centre third of the profile. In the case of "reservoir full" this condition was carried out absolutely, but for "the reservoir empty" a slight deviation from the condition was permitted, especially as the reservoir will never be entirely emptied.

The probable weight of the masonry was determined by an experimental block containing 4 cubic metres. After diminishing for 25 days the weight of the block remained constant at 2150 kilos. per cubic metre (134.23 lbs. per cubic foot), which weight was adopted in the calculations.

* Vth International Congress on Inland Navigation. Report by M. Gustave Cadart on the Reservoirs of the Department of the Haute-Marne.

The profile was determined by calculating the widths at horizontal sections two metres apart. Subsequently the pressures were determined on oblique sections from the foot of the up-stream facing and from other points.

The maximum pressure per square centimetre in the masonry is 6.58 kilogrammes (13,478 lbs. per square foot).

The top of the dam proper was made 3.50 metres wide and placed 2.05 metres above the highest water-level. As the dam was also to serve to carry a road 7 metres wide across the valley, the necessary additional width was obtained by constructing on the down-stream face of the dam a half viaduct 3.5 metres wide, consisting of 40 semi-circular arches of 8 metres span. The arches form 8 groups of 5 arches each, which are separated from each other by abutment-piers 2.80 metres thick. The ordinary piers have a thickness of only 1.80 metres at their lowest part.

The foundation trench was excavated at least one metre into solid rock, and considerably deeper for three anchor-walls.

The up-stream facing, formed of freestone coarsely prepared, was covered with three layers of burnt pitch and was afterwards whitewashed to prevent too great an absorption of heat.

CHAPTER IX.

DAMS IN ALGIERS.*

The Habra Dam (Plate XLIII.).—The great results obtained by irrigation in Spain induced the French government to encourage similar improvements in Algiers. How much the condition of agriculture in that country depends upon a good supply of water may be judged from the fact that the average rainfall is only about 15 inches, of which quantity, moreover, only one thirty-seventh reaches the streams. The rain is very unequally distributed over the different seasons, and the idea, therefore, naturally suggests itself to store the surplus water of the rainy months for the time of drouth.

Among the important reservoirs constructed by the French in Algiers for this purpose, the largest was that of the Habra River. Although the watershed of this stream contains one million hectares (3859 square miles), yet, owing to the climatic conditions stated above, the annual yield of water amounts to only 108,000,000 cubic metres (28,521,000,000 gallons). The variableness of the flow of the Habra River will be seen from the following figures:

	Litres.	Gallons.
Flow per second in summer,	500	132
" " " " winter,	3,000	792
" " " during great freshets,	700,000	184,898

The construction of the Habra reservoir was undertaken in 1865 by a private company, formed under a charter from the French government. According to the original plans, the desired storage capacity, which was fixed at 30,000,000 cubic metres (7,924,000,000 gallons), was to be obtained by closing the valley of the stream by a high earthen dam. Two failures, however, of similar works in the province of Oran (Algiers), one situated on the Sig at Tabia and the other on the Tlelat River, caused the projectors of the Habra reservoir to modify their plans by substituting a dam of masonry for one of earth.

The construction of the reservoir was commenced in November, 1865, but, owing to various delays, the work was not completed until May, 1873. After having been in successful use for about eight years, the Habra Dam was ruptured in December, 1881. This catastrophe, which occurred after an unusually severe storm, during which 6½ inches of rain fell in a very short time, caused the destruction of several villages, of part of the city of Perregaux, situated 6¼ miles from the dam, and the loss of 209 lives. The failure of this dam cannot be attributed to any defect in the design, but was caused, in all probability, by faults in the execution of the work.

The profile of the dam was determined by the method of M. Delocre, and consists, commencing at the top, of a rectangle and three trapezoids having the following dimensions:

* The dams marked T are taken from "Bacini d'Irrigazione," per G. Torricelli. Roma, 1885.

	Height.		Width.			
			Top.		Bottom.	
	In Metres.	In Feet.	In Metres.	In Feet.	In Metres.	In Feet.
1. Rectangle	6.00	19.68	4.30	14.10	4.30	14.10
2. Trapezoid	9.60	31.49	4.30	14.10	10.00	32.81
3. Trapezoid	10.00	32.81	10.00	32.81	19.10	62.65
4. Trapezoid	8.00	26.24	19.10	62.67	26.94	88.39
Total	33.60	110.22	....	....	....	....

A parapet 1.5 metres (4.92 feet) wide by 2.4 metres (7.87 feet) high surmounted the wall, preventing the waves from passing over its top, and serving as a foot-bridge.

What we have described above constituted the dam proper. It was founded entirely on rock, in the following manner: The irregularities of the rock surface were levelled with a bed of concrete, whose average depth was about 4 metres (13 feet). On this was laid a block of rubble masonry 2 metres high and projecting 2 metres beyond the front face of the wall. Upon this foundation the dam proper was built.

The main dam was straight in plan and had a length of 325 metres (1066 feet). It was flanked by an overflow-wall, 125 metres (410 feet) long, making an angle of 35° with its direction. The total length of the dam was therefore 450 metres (1476 feet), the top of the overflow being 1.6 metres (5.25 feet) below that of the main wall.

There were two scouring-galleries 35.7 metres (117.1 feet) apart, and having a cross-section of 1.2 metres wide by 2.24 metres high (3.94 feet by 7.35 feet) at the up-stream face, and of 1.5 metres wide by 4 metres high (4.92 feet by 13.12 feet) at the down-stream face. These galleries were closed by means of iron gates placed at their up-stream ends and worked from the top of the dam by means of rods and the proper gearing for hand-power. By opening the gates yearly it was thought that no deposits would form in their vicinity.

Water was taken from the reservoir by means of two outlets, each being composed of two pipes, 0.80 metre (2.62 feet) in diameter, passing through the masonry.

When the water was first allowed to fill the reservoir, the dam leaked to such an extent that it looked like a large filter. This loss of water ceased, however, in course of time.

The Habra Dam was finished in the winter of 1871–72, but on March 10, 1872, part of the overflow-wall failed during a severe freshet, on account of a defective foundation.

The plans of the Habra Dam and reservoir were prepared under the direction of M. Debrousse, C.E., President of the Society which constructed this work, and verified by M. Feburier, Consulting Engineer. M. Leon Pochet was in charge of the construction from 1869 to the end, and it is from the interesting memoir describing the work which this engineer published in the "Annales des Ponts et Chaussées" for April, 1875, that we have taken the description given above.

The causes which probably led to the failure of the main dam in December, 1881, are given very fully in the following:

Extract from a Memoir on the "Rupture of the Habra Dam" by Gaetano Crugnola, Ingegnere Capo Provinciale.*

"The construction of the dam began in 1866, and the work was finished in 1871. It was founded completely upon a kind of calcareous grit of the Tertiary epoch, which did not present everywhere the same consistency. Between two strata of hard grit which constitute the principal base of the dam there are others more or less soft, alternating with argillaceous strata which had to be removed at certain points to a great depth and were replaced by good concrete. Moreover, we must state, first, that the most important stratum of grit had a very limited depth, which, however, was considered sufficient to support the weight of the whole construction.

"Second. The plane of separation between the grit and the stratum of argillaceous schist of the Miocene period was not far distant, and had an inclination of 45° with reference to the horizon and towards the valley.

"Third. The strata of grit were inclined at 30° with the horizon.

"The material employed in the masonry had to be procured in the locality, as the construction of such a piece of work (which required 500 cubic metres for each lineal metre) was not possible except by using the building material indigenous to the valley. For so great a mass of masonry the materials had to be close at hand. Consequently, stones from the stratum of Tertiary grit upon which the dam was founded were used. It is important to know, in regard to the Habra Dam, that the strata of grit did not all present the same tenacity. Some had a very pronounced schistose structure, and, although the instructions of the 'Superior Administration' were clear and declared these stones defective, it cannot be assumed with certainty that none of this building material was used.

"The sand employed was not perfectly good. In the beginning of the construction it was taken from the Habra stream, but, when the dam reached a height above the ordinary level of the Habra, the water became stagnant and the quarries were filled with sedimentary deposits. It then became necessary to work some quarries at a greater distance from the place. The sand from these quarries was clean and free from loam, but too fine to make good mortar.

"Moreover, it is important to state that the 'Administration' itself had permitted the use of a red earth instead of sand for the inner part of the dam. Now the red earth contained an excess of clay, amounting to from 22 to 24 per cent of its weight. This is the reason why the mortar could not be relied upon to furnish the necessary resistance.

"The lime, although hydraulic, was not very good. It was made from calcareous rock found on the banks of the Habra River, which contained from 1 to 10 per cent of sand, and from 16 to 31 per cent of clay. For a construction which is destined to retain a column of water 34 metres high an eminently hydraulic lime should be employed, and it ought also to be kept in repose for a certain time before being used, in order to give the quicklime time enough to expand.

"It is known that all cements and hydraulic limes contain a certain quantity of quick-

* Published in the "Ingegneria e Arti Industriali di Pareto e Sacheri," Torino, 1882.

lime which does not expand immediately, but only after a certain time, so that the increase of volume of the cement causes porosity, if not actual cavities in the interior of the masonry.

This property of expansion was known to the French engineer Minard in 1827. From his experiments it appears that this expansion is not completed until twelve months after immersion, and sometimes not until after twenty-two months. This consideration is of great importance. If this expansion in the Habra Dam was on a large scale, it would evidently produce fatal consequences after a certain number of years.

Let us now examine the dam from another point, which will show more clearly the defects which probably existed in the construction. It is not possible to make a dam absolutely impermeable, and the result at "Furens," where only a few humid spots appeared on the outside face of the wall, is to be regarded as exceptional. These filtrations remained for a certain time, and then disappeared completely. In the Habra Dam, however, the filtrations were numerous. When the water reached a height of 10 metres, they appeared soon on the outside face. As the level of the water rose, the leakage increased to such an extent that the dam looked like a gigantic filter.

This phenomenon was attributed especially to the porous nature of the stones which were used. In the course of time the water of filtration deposited on the wall a thin, white, shiny stratum, which was a carbonate of lime like that of which stalactites are composed. This deposit was certainly derived from an excess of lime in the hydraulic cement, which was not transformed into a silicate, but remained dissolved in the water of filtration under the great pressure exerted by the liquid of the reservoir. On coming into contact with the air the lime became a carbonate and was deposited on the face of the wall.

From the above observations we see that the masonry was not suitable for this kind of construction, and that the cement would gradually lose its hydraulic and cohesive properties.

We have examined about all the circumstances which might have affected the stability of the construction, but cannot say definitely which of them caused the rupture of the dam, on account of not having some exact data with reference to the occurrence of that disaster. Nevertheless, we can say that the above-named circumstances, combined with the effect of the inundation of which we shall speak hereafter, caused the destruction of the dam.

The rupture was 100 metres (328 feet) long and 35 metres (115 feet) deep, going down to the base; from which it can be supposed that the foundations also may possibly have sunken. At any rate, the construction of the masonry, as regards the choice of materials, seems not to have been conducted with all the precaution which a work of such magnitude demands.

On the other hand, we ought to observe that the rupture occurred after a disastrous inundation, which was accompanied by very unfavorable meteorological conditions. The hydrographic basin which furnishes the water to the Habra reservoir has an extension of 800,000,000 square metres (309 square miles). In a very short time the height of the water resulting from the rain was observed with an udometer to be 0.161 metre (0.53 foot); and as the rainfall was general in the whole basin, the total quantity of water can be estimated at 128,800,000 cubic metres (34,021,361,000 gallons).

"As the evaporation could certainly not have amounted to much in such a short period of time, we can admit without exaggeration, keeping in mind the filtrations which would have been possible, that the dam permitted the passage in one night of more than 100,000,000 cubic metres of water (26,414,000,000 gallons).

"Now it is easy to understand that such an immense quantity of water would have flowed over the dam, forming a large wave whose height can be calculated at about 1 metre (the flow was about 5000 cubic metres (1,320,705 gallons) per second). As the breast-wall was 2.4 metres (7.87 feet) high above the ordinary level of the basin, the total superelevation of the water can be placed at 3.90 metres (12.80 feet).* Such an increase in the height of the basin could not, certainly, produce a notable change in the conditions of the stability as regards sliding. The pressure, however, on the exterior face would be remarkably increased. From an approximate calculation, taking into account the obliquity of the resultant, we have found that a superelevation of 1.50 metres (4.92 feet) above the ordinary level would be sufficient to cause pressures of 12 to 13 kilos. per square centimetre (12.29—13.31 tons of 2000 lbs. per square foot). From what we have said above, it will be seen that the superelevation was more than double the one we have assumed in the calculation, and that the masonry was consequently exposed in an extraordinary manner above the limits of safety, the conditions becoming still more unfavorable from the circumstance that the water overflowed the top like a gigantic cascade.

"In view of the catastrophes that may occur by constructing a reservoir, it will be asked if it is necessarily dangerous to accumulate such an immense quantity of water; but we reply without hesitation, No. The rupture of a dam which retains such a great volume of water will certainly be dangerous, especially if it is situated near populated places. But it can be asserted generally, that in the construction of such a wall we can follow all the laws depending upon its static conditions and the pressure of the water, which can be determined with certainty and without establishing any hypotheses which do not conform to the reality. If the masonry is carefully built, as it ought to be, both in the interior as well as in the points which join the bottom and lateral faces, no danger of rupture need be feared."

The Tlelat Dam[T] (Plate XLIV.) was built in 1869 on the Tlelat River to supply the village of Sante Barbe, situated at a distance of about $7\frac{1}{2}$ miles from its site, with water, and also for irrigation purposes. Its general dimensions are:

	Metres.	Feet.
Length on top,	99.0	324.80
Height,	21.0	68.90
Thickness at top,	4.0	13.12
" " base,	12.3	40.34

The maximum pressure in the masonry is 6 kilos. per square centimetre (6.14 tons of 2000 lbs. per square foot), and the back face has some tension. The capacity of the reservoir is 550,000 cubic metres (145,278,000 gallons), and the watershed supplying it contains 13,000 hectares (51 square miles).

* The height of the wave observed at the moment of the rupture was 3.50 metres (11.48 feet).

The back face of the dam is vertical; the front face is circular, having a radius of 40 metres (131.23 feet), the centre of the circle being 3.60 metres (11.81 feet) above the top of the dam. A parapet 1 metre high by 1.50 metres wide surmounts the wall. It has been decided to raise the dam 6 metres higher in order to obtain more storage.

The Dam of Djidionia[T] (Plate XLV.), located on the river of this name, was built in 1873–75 to supply the villages of St. Aimé and Amadema with water. The reservoir has a capacity of 2,000,000 cubic metres (528,282,000 gallons), and is supplied from a hydrographic basin containing 85,000 hectares (328 square miles).

The dam was built straight in plan. Its general dimensions are:

	Metres.	Feet.
Height above foundation,	17.0	55.78
" of foundation,	8.5	27.89
Thickness at top of dam,	4.0	13.12
" " base of dam,	11.5	37.73
" of foundation,	16.0	52.50

The maxima pressures are:

	Kilos per square centimetre.	Tons of 2000 lbs. per square foot.
Above foundation,	6.0	6.14
At base of foundation,	9.43	9.65

The inner face has a tension of more than 1 kilo. per square centimetre, but this does not seem to have injured the masonry.

The profile is bounded by a vertical line up-stream, and on the down-stream side by a straight line having a slope of 0.055 to 1 for a depth of 6 metres (19.69 feet) from the top, and then by a circular curve whose centre is 4.50 metres (14.76 feet) below the top of the dam and whose radius is 19 metres (62.34 feet).

It has already been decided to raise this dam 8 metres (26.25 feet), increasing thus the capacity to 5,000,000 cubic metres (132,062,000 gallons).

The Gran Cheurfas Dam[T] (Plate XLVI.), situated on the Mekerra River (Sig.) at a distance of about 9 miles from St. Dionigi, was constructed in 1882–84. Its general dimensions are:

	Metres.	Feet.
Length on top,	155	508.40
" at base,	50	164.04
Height above foundation,	30	98.42
Width at top,	4	13.12
" " base,	22	72.18

The dam is composed: (1) Of a foundation-mass of rubble 10 metres (32.81 feet) high, 41 metres (134.52 feet) thick at the base and 24 metres (78.72 feet) on top; (2) Of the wall proper, having both faces formed of parabolic surfaces.

The two parabolæ which bound the profile have the same axis, which is horizontal and at the level of the top of the dam. The vertices of the parabolæ are respectively at the front and back edge of the top of the profile. At the top of the foundation the up-stream parabola has an abscissa of 3.00 metres (9.84 feet), and the down-stream curve has an abscissa of 15 metres (49.21 feet).

The maximum pressure on the masonry is 6 kilos. per square centimetre (6.14 tons of 2000 lbs. per square foot), and the back face has some tension.

The capacity of the reservoir is 16,000,000 cubic metres (4,226,256,000 gallons). In 1885 part of the dam failed, the length of the breach being 40 metres (131.24 feet); but the dam has since been repaired.

The Dam of Hamiz[T] (Plate XLVII.) is situated on the Hamiz River at a distance of about 4½ miles from the village of Foundouk. It was built in 1885 to form a reservoir of 13,000,000 cubic metres (3,433,833,000 gallons) capacity. The watershed of this reservoir contains 14,000 hectares (54 square miles). The dam is built of rubble masonry, is straight in plan, and has both faces curvilinear.

The general dimensions of the dam are as follows:

	Metres.	Feet.
Length on top,	162.0	531.60
" at base,	40.0	131.24
Height above bed of river,	38.0	124.68
" " foundation,	41.0	134.52
Thickness at top,	5.0	16.40
" " base,	27.8	91.21

At a depth of 28.84 metres (94.62 feet) from the top the front face has an offset of 1 metre, the thickness of the dam at this point being 18.85 metres (61.85 feet).

The maximum pressure at the front face is 11 kilos. per square centimetre (11.25 tons of 2000 lbs. per square foot). The maximum tension at the back face is 3 kilos. per square centimetre (3.06 tons of 2000 lbs. per square foot).

The profile of this dam was determined by the French method of "equal resistance." *

* See page 2.

CHAPTER X.

VARIOUS EUROPEAN DAMS.*

The Dam of Cagliari [T] (Plate XLVIII.), situated on the Island of Sardinia at a distance of 13 miles from the city of Cagliari, was constructed in 1866 on the Corrongius River. The annual yield of this stream, derived from a watershed of 30,000 hectares (116 square miles), is estimated at 4,000,000 cubic metres (1,056,564,000 gallons).

The reservoir is 125 metres (412 feet) above the level of the sea, and has a capacity of 1,000,000 cubic metres (264,141,000 gallons). The principal dimensions of the dam are:

	Metres.	Feet.
Length on top,	105.0	344.50
" at base,	50.0	164.00
Height,	21.5	70.54
Width at top,	5.0	16.40
" " base,	16.0	52.50

This reservoir wall was founded on rock, and built of rubble masonry composed of granite stones and hydraulic mortar made of lime of Cagliari, and Pozzolona of Rome, mixed with well-washed granitic sand.

The Dam of Gorzente [T] near Alexandria (Plate XLIX.), was constructed in 1882 on the Gorzente River to form a reservoir of 2,835,000 cubic metres (748,840,000 gallons) capacity, from which the city of Genoa derives a supply of water through the "Deferrari Galliera" aqueduct.

The principal dimensions of the dam are:

	Metres.	Feet.
Length,	150.00	492.20
Height,	37.00	121.40
Width at top,	7.00	22.97
" " base,	30.35	99.57

The wall is surmounted by a parapet 1.50 metres high by 4 metres wide (4.92 feet by 13.12 feet), in which, however, three openings, each 2.50 metres (8.20 feet) wide, were left to form waste-weirs. A severe storm in 1885 showed these openings to be insufficient. The water rose 0.35 metre (1.14 feet) above the parapet and caused considerable damage to the front face of the wall.

The Gorzente Dam was founded entirely on rock, and was constructed of rubble masonry, composed of the serpentine stone found in that locality, and of a mortar made of lime of Casale and serpentine sand. The reservoir formed by this dam has a surface of 26 hectares (64 acres), and is supplied from a watershed of 1769 hectares (6.8 square miles).

* The dams marked T are taken from "Bacini d'Irrigazione," per G. Torricelli. Roma, 1885.

The Gileppe Dam* (Plate L.).—The reservoir in the valley of the Gileppe was constructed by the Belgian government to regulate the flow of this stream, and to furnish the important cloth manufactories at Verviers with a large supply of pure water.

M. Bidaut, the Chief Engineer who designed the Gileppe Dam and reservoir, commenced the preliminary studies in 1857, but, owing to various delays, his plans were not submitted to the ministry until 1868.

According to careful observations, the watershed of the Gileppe, containing 4000 hectares (9880 acres), yields from 20–23 million cubic metres (5,283,000,000, to 6,075,000,000 gallons) of water per annum. It was decided to construct a reservoir having a surface of 80 hectares (198 acres) and capable of storing 12 million cubic metres (3,170,000,000 gallons), by building a dam 45 metres high (147.6 feet) across the valley of the Gileppe. The same storage capacity might have been obtained by constructing four different basins having dams only 27 metres (88.6 feet) high, but the plan of one reservoir with a high dam was found to be more economical.

The Gileppe Dam was built curvilinear in plan, the radius being 50 metres (164 feet). Its greatest height is 47 metres (154.2 feet). The length on top of the wall is 235 metres (771 feet), at the base 82 metres (269 feet). The breadth of the wall is 15 metres (49.22 feet) on top and 65.82 metres (216.5 feet) at the base. The foundations were carried 1 metre into the rock. With the exception of a band of cut stones at the top and bottom of the front face, and at the angles where the batters change, the whole wall was constructed of rubble masonry, the total quantity amounting to 325,000 cubic yards. Although this massive dam was built with the utmost care, it was completed in six years. The work on the masonry progressed as follows:

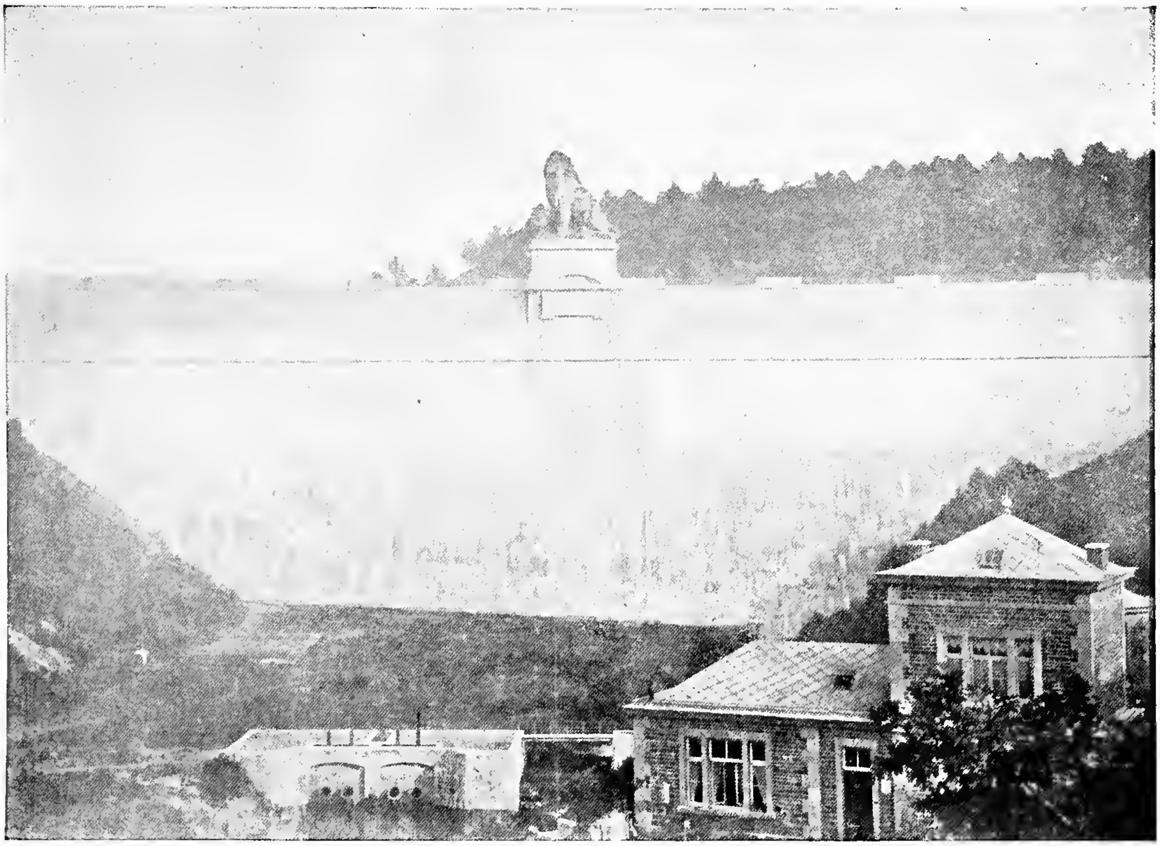
	Cubic Yards of Masonry.
1870,	19,400
1871,	78,500
1872,	59,300
1873,	68,900
1874,	46,500
1875,	52,400
	325,000

The average yearly work of over 54,000 cubic yards has probably never been surpassed in the construction of any other single structure. It was accomplished by 80 to 100 masons under the direction of 8 to 10 foremen. The work per man amounted daily to from 2.6 to 3.2 cubic yards.

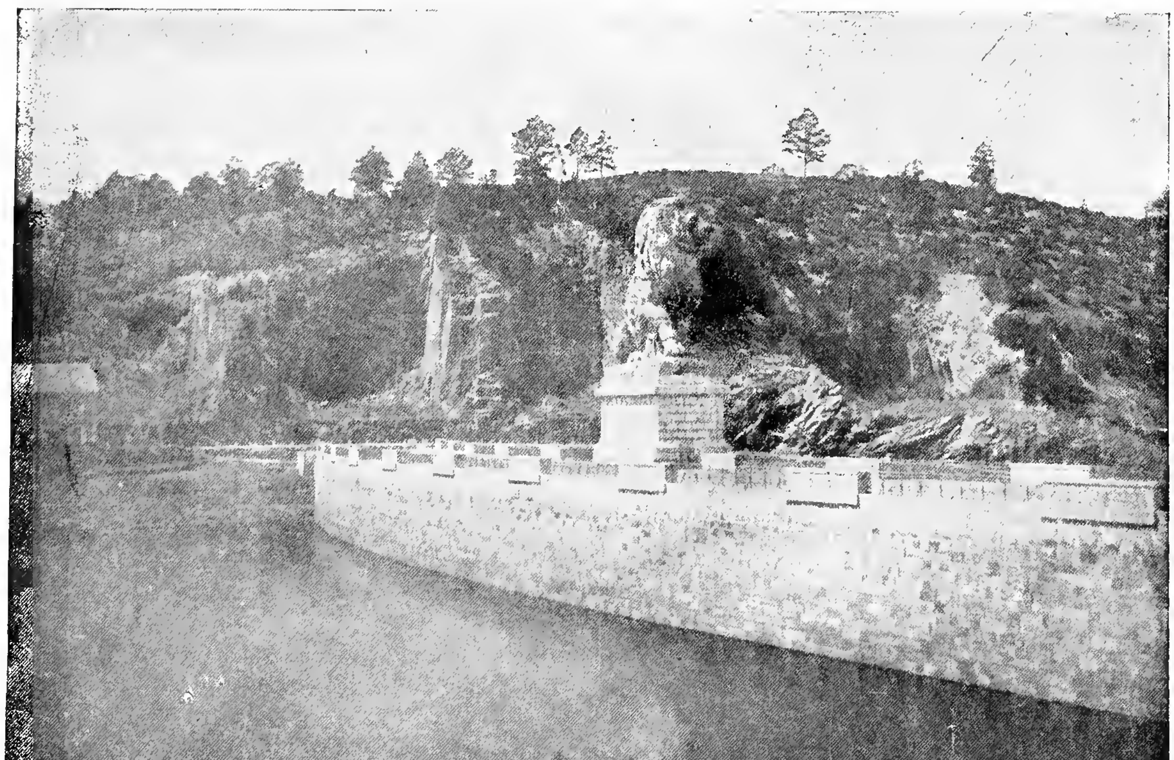
The sandstone or limestone used in the wall came from neighboring quarries, which were located at least 50 metres (164 feet) from the site of the wall and above the level of its crown. Two narrow-gauge railways served to transport the building materials to the dam.

Before commencing the foundations two subterranean channels were excavated, one on each side of the dam, by means of which the Gileppe was turned from its bed during the construction. These channels served subsequently as ways for the cast-iron outlet-

* Die Thalsperre der Gileppe bei Verviers. Von Ingenieur F. Kuhn. Published in "Der Civilingenieur," 1879.



GILEPPE DAM. (Front View.)



GILEPPE DAM. (Side View.)

pipes, by means of which water is drawn from two wells, each 2.8 metres (9.2 feet) diameter, placed in the reservoir.

Two overflow-weirs, 2 metres (6.58 feet) below the crown of the dam and 25 metres (82 feet) wide, situated one at each extremity, serve for letting the flood-waters escape. The carriage-road on top of the dam passes over the overflow-weir, ascending to the crown of the dam by grades of 1 in 7.

The leakage through the dam when the reservoir was first filled amounted to about 5300 gallons per day. This was probably due to the fact that the wall had not been exposed to the action of water during construction, as was done with the Furens Dam.

The leakage soon diminished; but even four years after the reservoir had been in use, a certain amount of moisture was perceptible on the down-stream face.

The total cost of the Gileppe Dam and reservoir was $874,000, amounting to 0.2 cent per cubic foot of storage room.

The profile of the Gileppe Dam has been severely criticised for its extraordinary top width of 15 metres (49.22 feet), and for involving about 75 per cent of useless masonry. It stands, indeed, in striking contrast to the scientific designs adopted for the dams of Furens, Ternay, and Ban, and resembles more nearly the early Spanish dams. To justify the great top width of the profile, it has been stated by the Belgian engineers that the dam was designed with a view of being raised to a greater height when more storage might be required. However, the main reason seems to have been a great timidity on the part of the Belgian engineers, who were fully impressed with the great body of water they were going to store (six times the contents of the Furens reservoir), and the calamity the failure of the dam would cause.

M. Bidaut, the Chief Engineer, went with his calculations of the stability of the dam even to the extreme of supposing water to percolate through the wall to such an extent that the specific gravity of the masonry would be reduced from 2.3 to 1.3.

The Vyrnwy Dam (Plate LI.) was commenced in 1882, and is still * in course of construction. It is to form a large storage reservoir on the Vyrnwy River for the water-supply of the city of Liverpool. This artificial lake, which is situated at a distance of 67½ miles from the old reservoirs at Prescott, will have an extent of 1115 acres, and an elevation of 825 feet above the level of the sea.

The Vyrnwy Dam will be 1350 feet long on top, the plan being straight. Its maximum height above the foundation will be 136 feet. The profile adopted differs from all others described in this book in not being designed simply to resist the water-pressure, but to form also a waste-weir. The front face of the wall is made, therefore, to conform to the curve described by the water in overflowing, and to deflect it into the basin in front of the dam.

The foundation is laid on a clay slate rock, which is frequently interspersed with hard volcanic ash, and ranges from a close-grained grit of dark bluish-gray color to a fine slate texture. The strata dip up-stream, the different beds varying in thickness and hardness. Great care was taken in preparing this rock for the foundation. All projecting portions or any parts which seemed in the least doubtful were removed.

* This description was written in 1887. The dam was finished in 1889.

The dam is being built of "Cyclopean rubble," which, owing to the great precautions taken, will have much greater strength than ordinary rubble. The stone used is of a similar kind to that excavated in the foundation. It weighs 2.06 tons per cubic yard, its specific gravity being 2.721. The quarry is situated at a distance of about one mile from the dam, and the stones are transported to the work by means of a double-track railway of 3 feet gauge.

The blocks are shaped roughly at the quarry. All thin, projecting pieces are cut off, and a flat but rough surface prepared for the lower bed. The best stones are reserved for the faces, and are cut to templates, their upper and lower beds being dressed parallel and their sides made vertical. An idea of the average size of the stones employed may be obtained from the following statement of the stones discharged from the quarry for the year ending October 18, 1885:

Stones under 2 tons, ...	45.99	per cent.
Stones 2 to 4 tons, ...	20.86	"
Stones 4 to 8 tons, ...	33.15	"

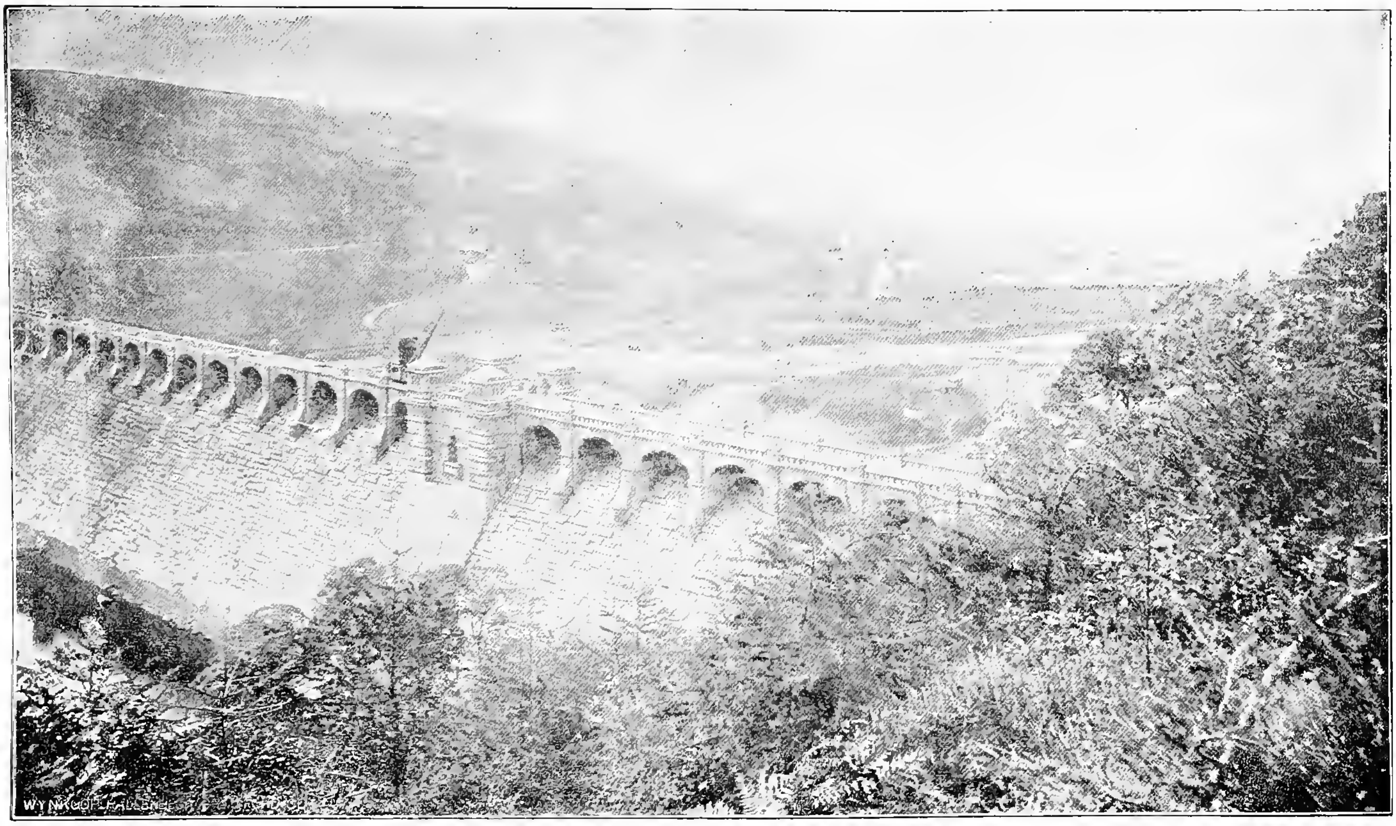
Before being placed in the wall, all stones, whatever their size, are scrubbed and subjected to jets of water under a pressure of 140 feet.

In the beginning of the work the sand for mortar and the gravel for concrete were obtained from the river-bed. As this material, however, contained a large amount of clay and oxide of iron, it was thoroughly washed in revolving cylinders having internal vanes arranged so as to lift and drop the sand and gravel. For the lower part of the dam the mortar was composed of two parts of this washed sand and one part of Portland cement. As the natural gravel after being cleansed still contained a large percentage of sand, the concrete was made of two parts of this gravel mixed with one part of Portland cement, without any further addition of sand.

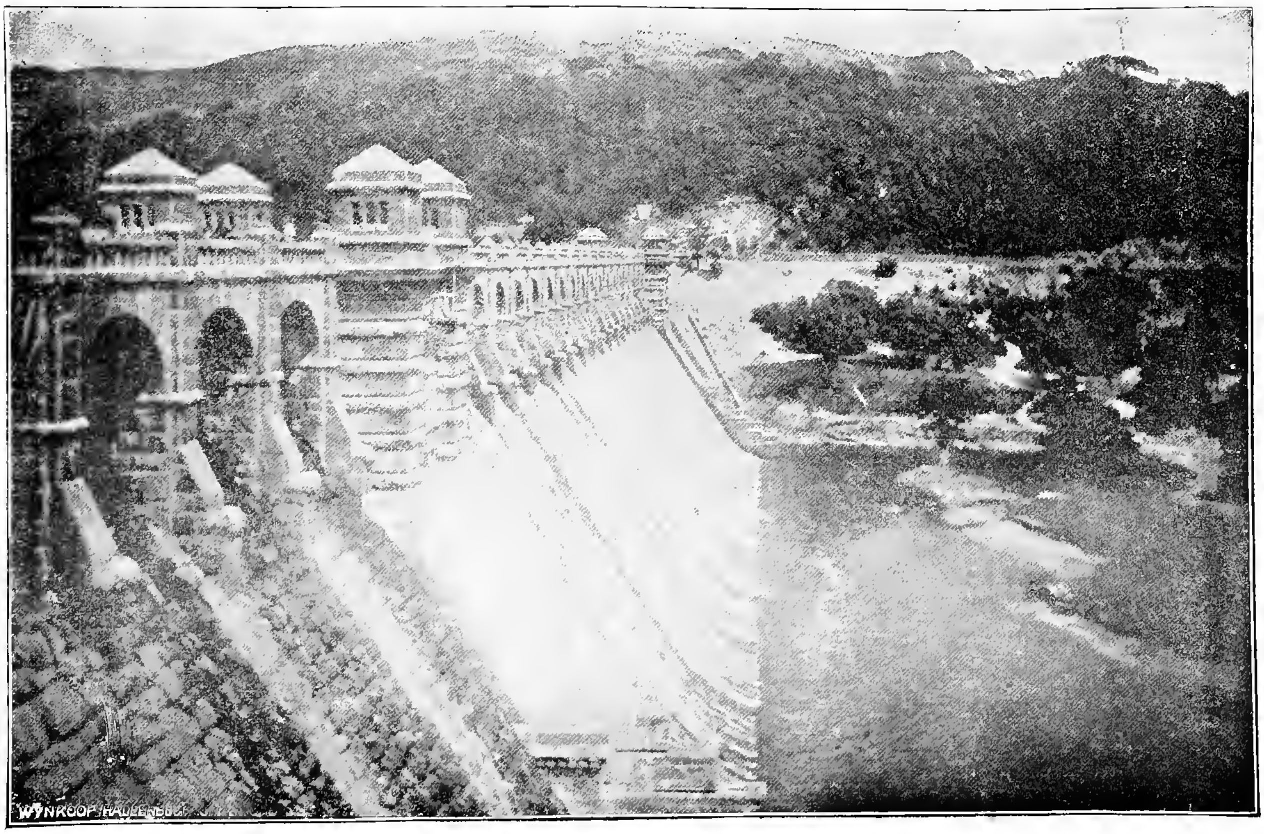
Experiments made in 1883 showed that by pulverizing the quarry-refuse-rock and mixing it with the natural sand in the proportion of two parts of the former to one of the latter, a stronger mortar was produced by using two parts of this mixture to one part of Portland cement than was obtained when only the natural sand was employed in a similar proportion. Since 1884 all the sand used has been obtained in this manner, and it has been found that the mortar produced from this mixture of sand has not only great strength, but also the very desirable quality of "an absence of shortness."

All the mortar used in the dam is made with Portland cement which is required to stand the following test for tensile strength: Of six briquettes, 8 days after being moulded, kept in water from the 2d to the 7th day, at least one must sustain without fracture a tensile strain of 5 cwts. per square inch for one hour. The average strength of about 9000 briquettes tested in this manner has been $6\frac{1}{8}$ cwts. As regards fineness, it is specified that not over 10 per cent of the cement shall be retained by a sieve having 60 brass wires to the lineal inch and weighing $3\frac{3}{4}$ ounces per square foot.

It is well known that Portland cement of great strength may be obtained by using a large amount of chalk in its manufacture; but unless the cement is burnt thoroughly it will contain lime in an uncombined state, which when mixed with water slakes and



Vyrnwy Dam.



Vyrnwy Dam.

swells, contracting subsequently when the surrounding cement is just acquiring its hardness. Most Portland cements have some free lime, which, however, owing to its great affinity for moisture, may be converted into a harmless hydrate of lime by merely exposing the cement to the air. To effect this purpose all the cement used for the Vyrnwy Dam is spread, six inches thick, on platforms placed one below the other and 18 inches apart. Each platform consists of loose boards which can be turned so as to drop the cement on the platform below. In this manner it is exposed seven times, being left on each platform one or two days, depending upon the dampness of the air. Owing to these precautions taken with the cement, no "hair-cracks" have appeared in the mortar used in the work.

The sand and cement are mixed dry in accurate proportions in revolving cylinders having internal vanes. Before passing out they are wetted uniformly by passing through a water-spray. Originally the sand and cement were mixed 2 to 1, but since April, 1884, this proportion has been changed to $2\frac{1}{2}$ to 1.

The dam is being built in the following manner: A level bed is first prepared on the rock, or on the masonry already laid, and is covered with a two-inch layer of cement mortar, which is beaten to free it of air. A large stone is then lowered into position by a steam crane, and is beaten down into the mortar by blows from heavy hand-malls. Other large stones are similarly placed, but so as not to touch each other. The spaces left between them are filled either with rubble made with small stones, or with concrete which is thrust into the narrow spaces with blunt swords. The work within the reach of each crane is brought up 6 to 8 feet before the crane is moved. In each course the large stones are laid so as to bond with those in the course below. There are no horizontal joints passing through the wall, as the top of each course is left with projecting stones and hollows, which permit it to be well bonded with the next course. To make the back face thoroughly water-tight, the vertical joints for several feet from the face are filled with mortar alone into which broken stone is forced.

Seven steam cranes are used in the construction of the dam, each with its driver and 18 men laying on an average 40 cubic yards per day. The specific gravity of the masonry, based upon the actual weights of the materials used up to the end of 1885, was found to be 2.577. Numerous tests made with 9-inch cubes of concrete taken directly from the wagons as it went into the work showed the crushing strength of the concrete when one year old to be about 187 tons per square foot.

The area of the typical section shown in Plate LI. is 8972 square feet. When the reservoir is empty and the front face of the wall is subjected to a normal wind-pressure of 40 lbs. per square foot, the maximum stress on the masonry amounts to 8.7 tons per square foot. When the reservoir is full and the wind is blowing down the valley with a force of 60 lbs. per square foot, the maximum stress on the masonry is 6.36 tons, and the angle made by the resultant pressure with a vertical line is $16° 39'$.

To prevent the possibility of a greater upward water-pressure under the dam, in case the foundation should prove to be pervious and the dam impervious, than that due to the 47 feet of water in front of the wall, a complete system of drains was constructed in the foundation. Where the bed-rock has the lowest elevation there are twenty-six such drains in a length of 198 feet of the wall. They are 9 to 12 inches square, and lie on the

rock near such places where leakage is apt to occur. The drains are kept 25 feet from the front face and 30 feet from the back face, and connect with a central tunnel, 4 feet high by 2′ 6″ wide, which traverses the foundation longitudinally at an elevation of 46.5 feet above the base of the typical cross-section. By means of a cross-tunnel leading down-stream any water that may filter into the foundation above the elevation of the drains is discharged at the front face of the dam.

The description we have given above has been taken from the "Report of Mr. George F. Deacon, C. E., as to the Vyrnwy Masonry Dam," made to the Water Committee of the city of Liverpool in December, 1885.

The Remscheid Dam* was built in 1889 to 1892 across the Eschbach Valley, to form a storage reservoir of 35,310,500 cubic feet capacity for the water-supply of Remscheid, Germany. The plans for the work were made by Prof. O. Intze. The dam is about 82 feet high, and is 13 feet 1½ inches wide on top and 49 feet 2½ inches at the base. It is curved in plan to a radius of 410 feet. The profile is designed to keep the lines of resistance within its centre third, reservoir full or empty.

The dam contains about 617,935 cubic feet of masonry, weighing about 4045 pounds per cubic yard. The stone used is a hard Linneite slate, quarried near the dam. It has a specific gravity of 2.7. About 38 per cent of the masonry consists of mortar, composed of 1 part lime, 1½ parts powdered Trass, and 1 part sand. This mortar sets much more slowly than one made with cement, and can be left mixed a whole day without injury. To make the dam as water-tight as possible, the back face was plastered first with cement mortar and then with asphalt. A brick wall (1½ to 2½ bricks thick) was laid on top of the layer of asphalt, cement mortar being used. The dam has proved to be perfectly water-tight.

The Einsiedel Dam† was built in 1890 to 1894 to form a reservoir storing about 95,000,000 gallons for the water-supply of the city of Chemnitz, Germany. The dam is 590 feet long on top. Its greatest height is 65.6 feet above the natural surface and about 92 feet above the foundation. The dam is 13.1 feet wide on top and 65.5 feet wide at the lowest foundation. In plan the wall is curved to a radius of about 1310 feet.

The dam was built of "cyclopean rubble," the stone used being hornblende slate, quartzite slate, and clay slate. The mortar consisted of 1 part cement, ½ part fat lime, and 5 parts washed sand. About 31,600 cubic yards of masonry were laid in the dam, about one third of the contents being mortar.

The waste-weir is 82 feet long. Water can be drawn from the reservoir through three gates placed at different elevations in the side of a gate-house built of concrete on the up-stream face of the dam. The outlet- and waste-pipes pass through a culvert in the dam, the inner end of which is closed by a masonry bulkhead. Stop-cocks placed in a vault at the lower face of the wall serve to control the flow through the pipes.

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* Engineering News of 1896.

† Engineering Record of 1894.

CHAPTER XI.

DAMS IN ASIA AND AUSTRALIA.

The Poona Dam* (Plate LII.) was constructed to form a large reservoir, Lake Fife, on the Mutha River, 10 miles west of Poona, for irrigating a large district of land near Poona and also for furnishing that place with a water-supply.

This project was first proposed by Col. Fife, R.E., in 1863, but the final plans were not approved and the work commenced until the latter part of 1868.

The dam was founded on rock and constructed entirely of uncoursed rubble. Its maximum height is 98 feet above the river-bed and 108 feet above the foundation. Its length is 5136 feet, 1453 feet of which form the waste-weir, whose crest is 11 feet below the top of the dam.

The line of the dam is formed of different tangents. At their intersections the wall is reinforced by heavy buttresses of masonry. The top-width of the dam, on the different tangents, varies according to the height of the structure.

As the masonry showed signs of weakness after being completed and subjected to water-pressure, it was reinforced by an earthen bank, having a top-width of 60 feet and a height of 30 feet, which was constructed against the lower face of the dam. The total cost of the structure was $630,000.

The dam backs up the water for 14 miles and forms a reservoir having a capacity of 3,281,200,000 cubic feet and a surface of 3681 acres. Only [illegible]9 feet of depth of water is available, owing to the elevation which had to be given to the canals that are fed from the reservoir.

The catchment basin, above the dam, contains 196 square miles on which the annual rainfall is about 200 inches.

The Tansa Dam† (Plate LIII) was constructed to form a large reservoir for the water-supply of Bombay. This project was first proposed by Major Hector Tullock, R.E., in 1870. The final plans were prepared by Mr. W. Clerk, who has had full charge of the construction of the works as Executive Engineer.

The total length of the dam is 8800 feet and its maximum height above the foundations is 118 feet. The structure is, however, designed sufficiently strong to permit of its height being increased 17 feet, in which case its length would be 9350 feet.

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* Irrigation in India, by Herbert M. Wilson (see XIIth Annual Report, U. S. Geological Survey).

† See Engineering News, June 30, 1892.

" Engineering Record, December 19, 1891.

" XIIth Annual Report of the U. S. Geological Survey. Irrigation in India, by Herbert M. Wilson.

1650 lineal feet of the dam forms the waste-weir, the crest of which is 3 feet below the top of the dam.

The reservoir has an area of 8 square miles. The available depth of water above the sill of the discharge-sluices is only 20 feet. The net available capacity of the reservoir is 691,300,000 cubic feet, but this might be much increased, if necessary, by placing the sluice-gates lower.

The catchment basin above the dam is only $52\frac{1}{2}$ square miles. Owing to the steepness of the slopes and a rainfall of 150–200 inches per annum, the daily discharge from this basin is estimated at about 8,000,000 cubic feet per day.

The site of the dam is in a dense forest and jungle. To carry on the work a village had to be built for the native workmen and a macadamized road, 8 miles long, had to be constructed to the nearest railroad station.

The dam was founded throughout on rock, the excavation for the foundations being in places 45 feet deep. Its alignment consists of two tangents, located so as to make the excavation to the bed-rock as little as possible.

The dam was constructed entirely of uncoursed rubble masonry, roughly scabbled on the facing. The stones used were hard trap or greenstone, in pieces which could be carried by two men.

An excellent hydraulic lime was obtained from the nodules of limestone, called kunker, which are found in the ground in abundance. Most of the cement used in India is obtained from these kunkers, which are generally about the size of a man's fist, although in the Ganges Valley they are found in blocks weighing 100 pounds or more. They are found in the clay deposits, which are very abundant in India.

The cement was burnt at the site of the dam. Kunker nodules were excavated some feet below the surface of the ground, exposed to the sun, dried, beaten, and washed clean, before being burnt.

The sand used, clean, sharp trap or quartz, was carefully washed before being mixed with the cement.

Some idea of the magnitude of this piece of construction can be formed by the following items of work performed:

Total excavation,	251,127	cubic yards.
Loose rubble-stone,	544,700	" "
Lime,	81,700	" "
Washed sand,	122,555	" "
Rubble masonry,	408,520	" "

The proportion of the mortar (consisting of 1 part cement to $1\frac{1}{2}$ parts of sand) in the masonry was found by a careful calculation to be $36\frac{7}{10}$ per cent. In the lower part of the dam some Portland cement was used.

The largest amount of masonry per month was laid in January 1891, when 700 masons laid 26,000 cubic yards of rubble.

During the working season (May to October), 9000–12,000 men were employed on the work.

During the rainy season the work had to be suspended.

The construction was commenced by the contractors, Glover & Co., in March 1886, and completed in April 1891, 15 months ahead of the contract time.

The cost of the dam was about $1,000,000.

The masonry has proved to be perfectly water-tight.

The Bhatgur Dam* was constructed on the Yeluand River, about 40 miles south of Poona, in the Presidency of Bombay, to form a large reservoir for irrigation purposes. The uncertainty of the rainfall in a portion of the Poona collectorate led Col. Fife, R.E., in 1863, to make surveys to find some means of supplying this region with water. This work was soon discontinued, but resumed subsequently by Mr. J. E. Whiting, C.E., and continued to 1871, when the final plans were decided upon.

The works were carried out under the direction of Mr. Whiting. They consist of the Bhatgur reservoir, having a capacity of 5,510,740,000 cubic feet, of the Nira canal, 129 miles long, and of a diversion-weir, at the head of the canal, 19 miles below the reservoir site.

The reservoir is formed by a masonry dam having the following general dimensions:

Length of dam, ...	4067 feet.
Maximum height above foundation, ...	130 "
Top-width, ...	12 "
Bottom-width, ...	73.7 "

The maxima pressures in the masonry are:

At down-stream face, ...	5.8 tons per square foot.
At up-stream face, ...	6.7 " " " "

The profile of the dam was determined by a modern formula, similar to that of M. Bouvier's.

The catchment basin above the dam contains 128 square miles.

Waste-weirs, having a total length of 810 feet, are constructed in the body of the main dam, at both ends. They can pass a depth of water of 8 feet. The roadway is carried over the weirs by a series of arches having spans of 10 feet.

To pass the floods, which amount, at times, to 50,000 cubic feet per second, there are, in addition to the waste-weirs, twenty under-sluices, 4 × 8 feet in area, having their sills 60 feet below high-water mark.

With this great head, the sluices can discharge 20,000 cubic feet per second, the average flood.

The sluice-openings are lined with the best ashlar masonry, and are closed by iron gates, which slide vertically and are operated by steel screws, worked from the top of the dam by a female capstan-screw turned by hand levers.

* XIIth Annual Report of the U. S. Geological Survey. Irrigation in India, by Herbert M. Wilson Engineering Record, December 19, 1891, and July 30, 1892.

The main object of the sluices is to discharge the water from the reservoir into the river in which it flows about 20 miles to the diversion-weir at the head of the Nira canal. A less number of sluices would have been snfficient for this purpose, the object of having so many being to prevent the silting up of the reservoir. This can only be accomplished by keeping all of the sluices partially open, when the river carries much sediment.

In this connection, Mr. A. Hill, the superintending engineer, states:

"Scouring sluices have little effect unless the area of the openings is great compared to the area of the floods. To remove silt already deposited they are useless, as has been proved by the manner in which they have silted up at Lake Fife and at Vir and other places where their area is small compared with that of the area of the floods. At Bhatgur they are intended not to remove silt deposited already, but to prevent its deposit by carrying it off while in suspension. If the dam is high and the discharge of the under-sluices will keep the flood level below the full-supply level, then they will be efficient. If the dam is low and the sluices will not keep the flood level below full-supply level, they will have little effect."

Automatic sluice-gates 8 feet by 10 feet, patented by Mr. E. K. Reinold, are to be placed on the waste-weirs. They will be arranged so as to be wide open when the floods reach a level of 8 feet below the crest of the dam, and to close gradually as the water lowers.

The Betwa Dam* has been built recently across the valley of the Betwa River, an affluent of the Jumna River, in India, to divert its water into an irrigation canal, and to form a large storage reservoir, having a capacity of 1,603,000,000 cubic feet.

Water is supplied in this manner to about 150,000 acres of land, which are contained in a region which has an annual rainfall of only 35 inches.

The Betwa project was first proposed by General Strachey in 1855. It was investigated by various engineers from time to time, but the plans were not finally approved by the Government until 1873.

The flow of the Betwa River varies from 50 cubic feet per second to 750,000. To pass the large quantity of water in time of freshets, the whole dam was built as an overfall weir. It was skilfully located at a wide part of the river, where a rocky ledge offered a good foundation.

The total length of the dam is 3296 feet, its height varying from 0.4 feet to 60 feet. The plan is convex up-stream. Two islands divide the weir into three parts.

As originally proposed, the dam was to have a top-width of 10.5 feet and a slope on both faces of 10 feet horizontal for $25\frac{1}{2}$ feet vertical. The Chief Engineer, Col. Greathed, changed the plan, however, so as to make the top-width 15 feet and the down-stream face nearly vertical, so that 6 inches depth of water would pass over the weir without falling on the face of the dam. The dam was made excessively strong, its bottom-width being greater than its height.

A water-cushion was formed in front of the weir, by building a subsidiary dam,

* XIIth Annual Report of the U. S. Geological Survey. Irrigation in India, by Herbert M. Wilson.

having a maximum height of 18 feet, about 1400 feet down-stream from the main structure, across the channel of the river.

Below the water, thus backed up against the dam, a large block of masonry 15 feet wide by about 20 feet high was constructed in front of the dam.

The body of the dam was built of rubble masonry, coursed at both faces and laid in native hydraulic-lime cement. The coping was made of granite ashlar, 18 inches thick, laid in Portland-cement mortar.

The dam is provided with suitable sluices for scouring the reservoir and for controlling the flow into the canal.

The Periar Dam (Plate LIV)* is being constructed across the Periar River, in the Province of Madras, India, to form a reservoir of about 13,100,000,000 cubic feet capacity for irrigating purposes. The water is to be conveyed into the valley of the Vigay River by means of a tunnel 6650 feet long, having an area of 80 square feet, and is to be used for irrigating about 140,000 acres of land.

The project of diverting water from the Periar River to the Vigay Valley was considered as early as 1808 by Sir James Caldwell, who rejected the scheme as unworthy of consideration.

In 1867 Major Ryves revived the project and made a report recommending the construction of an earthen dam, 162 feet high, across the Periar River. Colonel Pennycuick, who was given full charge of the project in 1868, proposed the construction of a masonry dam instead of one of earth, and worked out the details of the plans that were finally adopted. The cost of the reservoir, tunnel, and auxiliary works was estimated at $3,220,000.

The construction of the masonry dam was commenced in 1888 and has not yet (June 1893) been completed. The profile adopted for the dam is shown on Plate LIV. It was based on the conditions that the lines of pressure, reservoir full or empty, should be kept within the centre third of the profile, and that the maxima pressures, at the back or front face of the dam, should never exceed 18,000 lbs. per square foot, calculated by M. Bouvier's formulæ.

Owing to the difficulty experienced in obtaining skilled masons, it was determined to build the dam of concrete formed of 25 parts of hydraulic lime, 30 of sand, and 100 of broken stone. Both faces of the dam, however, were to be constructed of uncoursed rubble.

The ingredients of the concrete are mixed mechanically by means of turbine wheels. The lime used is obtained from a quarry situated about 16 miles from the dam, and is of excellent quality, being about equal to the well-known "Theil lime" which was used for the large masonry dams near St. Etienne, France.

Good, sharp, syenitic sand is found in the river-bed.

Most of the stone for the masonry is obtained from the excavations made in connection with this work. About 185,000 cubic yards of masonry will be required for the dam.

There are to be two waste-ways, one on each bank of the river, for which depressions in the hills will be used. Their aggregate length will be 920 feet.

* This description was written in 1893. The dam was completed in 1897.

The construction of the Periar Dam involves unusual difficulties, the work on the foundations having been limited to the three dry months January, February, and March. From the end of May to the beginning of December the ordinary flow of the Periar River is about 4000 cubic feet per second. In January the discharge commences to diminish very much and amounts to only about 250 cubic feet per second. During February and March it is even less. The drainage area back of the dam is 300 square miles, on which annually 65–200 inches of rain fall, the average being about 125 inches. The depth of water flowing off the shed is about 49 inches yearly.

But the flow of water in the river is not the only difficulty to be overcome. From the end of March to the beginning of June the malaria is deadly. Some idea of the difficulties involved in this construction may be formed from the following extract of a letter written by Colonel Pennycuick, the Chief Engineer of the work, to the writer in April 1890:

"The peculiarity of this work is not so much the actual height of the dam (173 feet), as the combination of height with the size of the river, the discharge of which rises to over 120,000 cubic feet per second at times, and the peculiar conditions of the site, which is in a jungle inhabited by nothing but elephants, bison, and tigers, seventeen miles from the nearest habitation, and eighty-three from the nearest railway station. Labor has to be paid for at appalling rates, and the country bears a bad name for fever, of which the natives are much afraid. We have, in fact, to stop work entirely on that account for three months, and those the best of the year for river work; add to this that except in February and March you cannot be certain of a fortnight, at a time, without a flood, and that it is an every-day occurrence for the discharge to rise in a few hours from 500 cubic feet per second to 4000 or more, and you will understand that putting a dam across a river of this kind is not an easy job."

For a full account of how the difficulties involved in building the foundation of the Periar Dam were overcome we refer the reader to a series of articles on this subject by A. T. Mackenzie, A.M.I.C.E., in "Engineering" for 1892, and to a condensed account in "Engineering Record" of December 31, 1892. See also "the XIIth Annual Report of the U. S. Geological Survey. Irrigation in India, by Herbert M. Wilson."

The Beetaloo Dam,* in South Australia, was constructed in 1888–1890 to form a reservoir of 800,000,000 imperial gallons' capacity for impounding water for the domestic water-supply and irrigation needs of a district of 1715 square miles, including several towns.

The water is distributed by 255 miles of pipes 2–18 inches in diameter.

The dam was constructed entirely of concrete mixed by machinery, the total quantity required being about 60,000 cubic yards.

* Engineering News, May 30, 1891, and September 19, 1891.

Its general dimensions are as follows:

Maximum height, ...	110 feet.
Width at top, ...	14 "
" " bottom, ...	110 "
Length at top, ...	580 "

Spillway 200 feet long by 5 feet deep.

The plan was curved to a radius of 1414 feet. The profile adopted for the dam was Prof. Rankine's Logarithmic type (see Plate III). The masonry was founded entirely on rock.

The reservoir formed has a length of 1¼ miles, an average width of 530 feet, and a depth at the dam of 105 feet.

The work was begun in February 1888 and completed in October 1890, the total cost being $570,000. Mr. A. B. Moncrieff was Chief Engineer.

The Geelong Dam * (Plate LV.).—This dam was built across the valley of Stony Creek to form a reservoir for the water-supply of Victoria, Australia. It is built on a curvilinear plan, the radius of the vertical part of the back face being 300 feet.

The masonry consists entirely of concrete, as it was thought to be cheaper than rubble, and also to form a more perfect monolith. The concrete was made of broken sandstone, mixed in a puddling-mill with hydraulic mortar which was composed of Portland cement and pit-sand. The best results were obtained by mixing the ingredients in the following proportions:

2″ stone, ...	4½ parts.
Screenings, ...	1½ "
Sand, ...	1½ "
Cement, ...	1 "
Total, ...	8½ parts.

The stone of which the concrete was made weighed about 163 pounds per cubic foot. The average weight of the concrete was 143 pounds per cubic foot.

The cement and sand were mixed dry, then made into mortar and thrown over the broken stone. Great pains were taken to place the concrete before the cement commenced to set. The work was carried up in courses a few inches thick, each course being rammed until the mortar flushed the surface. Before commencing a new course the surface of the preceding one was well watered and mopped over with cement grout immediately in advance of the new concrete.

The Geelong Dam is coped with heavy blue stones, which are 3′ 3″ wide by 1′ 9″ deep. Although waves four feet high break over the top of the dam, not the slightest damage is apparent.

Two pipes, 24″ diameter, pass through the dam: one serves for the "outlet," and

* Proc. Inst. C. E., vol. lvi., p. 93.

the other, which is placed at a lower level, to scour the reservoir. Both pipes have stop-cocks on the down-stream side of the dam.

When the Geelong reservoir was first filled a little water found its way through the dam; but this leakage soon stopped, owing to hard incrustations of lime being formed on the dam.

The Tytam Dam (Plate LVI.)* is being constructed near Hong Kong for the Tytam Water Works. The foundation was laid on decomposed granite and boulders, as solid rock could not be found without going to a great depth. Owing to the difficulty of securing skilled masons for this work, it was decided to build the dam of stones about 3 to 6 cubic feet in size, laid in a matrix of concrete. The wall is being constructed in the following manner:

The inner face is composed of ashlar masonry of granite, laid in courses one foot high with plenty of headers. The side joints of the stones are grouted, the mortar being composed of 1 part Portland cement to 2 parts of sand. Next to the inner face 2 feet of "extra fine" concrete, composed of 4 parts of stone (1″ cubes), 6 parts of sand, and 2½ parts of Portland cement, is placed in order to form a water-tight skin. Next comes 5 feet of "fine concrete," composed of 4½ parts of stone, 3½ parts of sand, and 1 part of Portland cement. The "hearting" of the wall consists of "fine concrete" mixed as above, with stones 3 to 6 cubic feet in size imbedded in it. These stones are not placed closer than 3 inches from each other, the spaces between them being filled with concrete, which is well rammed. The outer face is carried up in steps for convenience of getting across the valley. It is not being built exactly as shown in Plate LIV. The facing is only one stone thick, and has its back irregular so as to bond with the concrete.

The dam is being constructed in courses about 2 feet thick, and inclined up-stream so that the front face is about 2½ feet higher than the back face. Each course has plenty of projecting stones to bond it with the next one.

After the outer facing and the rubble-concrete hearting of a course have been laid, the "fine concrete" is placed between this masonry and planks. The back face is then carried up, and finally the "extra fine" concrete is rammed between this face and the "fine concrete." About three fifths of the bulk of the whole wall will consist of concrete, and two fifths of stone. The best London Portland cement is used, about one barrel of cement being required per cubic yard of masonry.

The stone for the concrete is crushed to pass through a 1¾″ hole by machinery, and is mixed with the mortar in revolving cylinders. The screenings from the "crusher" are used as sand, this article being very scarce. The river sand used is not washed, as the clay, or rather decomposed granite, it contains, is considered an advantage as conducing to water-tightness. For the same reason the engineers in charge of this work use plenty of sand in the concrete, and take care not to have it too sharp or clean, the object being not so much to obtain strength as to make the mortar impervious.

The foundation of the dam was prepared in the following manner: After cleaning the surface of decomposed rock and boulders thoroughly, liquid cement mortar, mixed 3 to 1, was spread over it; then 3 inches of stiffer mortar was placed. Before this mortar could dry, 18 inches of "extra fine" concrete was laid, then 3 feet of fine concrete, and finally the rubble blocks.

* This description was written in 1887.

To allow any water that might leak into the wall to escape freely, perforated zinc pipes 1½ inches diameter were placed in the masonry about 5 feet apart, and later on only bamboos; but this precaution was hardly necessary. When the water had risen to the top of the fourth step, the leakage could be carried off by a one-inch pipe without pressure.

The water will be taken from the reservoir by means of a valve-well having inlets at different elevations. The well is placed at the centre of the dam, which is reinforced at this point by a pier. A cast-iron midfeather divides the well into halves, one being full of water and the other dry. The valves are placed in the dry part.

The description of the Tytam Dam which we have given above has been taken from a letter of Mr. James Orange, the engineer in charge of the work, addressed to Mr. B. S. Church, Chief Engineer of the New Croton Aqueduct, to whom we are indebted for this information.

The Toolsee Dam.* was built according to the logarithmic profile designed by Prof. Rankine (see Plate III), its total height above bed-rock being 79 feet. It forms a lake for the water-supply of Bombay.

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* Spon's Dictionary of Engineering, Vol. VIII, p. 2743.

CHAPTER XII.

AMERICAN DAMS.

The Boyd's Corner Dam* (Plate LVII.) was constructed on the west branch of the Croton River, to form a storage reservoir having a capacity of 2,722,720,000 gallons for the city of New York. The reservoir has a surface of 279 acres, the maximum depth of the water being 57 feet. The general dimensions of the dam are:

Length on top, ...	670.0 feet.
" at level of the river, ...	200.0 "
Maximum height above foundation, ...	78.0 "
Width at top, ...	8.6 "
" " base, ...	57.0 "

Plate LV. shows the profile of the dam as designed by the Chief Engineer, Geo. S. Greene. It was built with cut-stone facings, and a hearting of concrete into which large stones were placed from the base to 15 feet above the stream. The concrete was mixed in the proportion of $4\frac{1}{2}$ parts of stone, 2 of sand and 1 of cement. It weighed $133\frac{1}{4}$ pounds per cubic foot.

Water is drawn from the reservoir by means of a tower having two 36-inch outlet-pipes which pass through the dam. The overflow is about 100 feet long, and was formed by excavating the rock at the north-east end of the dam.

The work was done originally under the direction of "The Croton Aqueduct Board." However, in 1870, when the dam was almost completed, the control of the work was transferred to the Department of Public Works. The new authorities changed the plans by building against the up-stream face of the dam an earthen bank 20 feet wide on top and having a slope of 5 to 2. According to Mr. J. J. R. Croes, the engineer in charge of the construction of the dam, this embankment was built of porous material which would not puddle well. "It was built by contract, and not rolled or thoroughly rammed, but merely carted over."

Under these circumstances the earthen embankment must have become saturated, subjecting thereby the dam to an increased pressure instead of reinforcing it.

The work was commenced in September, 1866, and completed in the fall of 1872. The masonry dam contains about 21,000 cubic yards of concrete and 6000 cubic yards of cut stone.

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* See "Memoir on the Construction of a Masonry Dam," by J. J. R. Croes, C.E., in the Papers of the American Society of Civil Engineers for 1874.

The Bridgeport Dam * (Plate LVIII.). This dam was built across the Mill River at a point $5\frac{1}{4}$ miles from Bridgeport, to form a new storage reservoir for the water-supply of that town. The general dimensions of the dam are:

Length on top, ...	640 feet.
" at bottom of stream, ...	50 "
Maximum height, ...	40 "

The west end of the dam forms an overflow-weir 80 feet long, being 5 feet below the guard-wall.

The scouring-gallery is 3 feet 4 inches by 3 feet 4 inches in the clear, and is closed by a suitable gate, which is operated by worm gearing.

A gate-chamber 10 feet by 15 feet in the clear, lined with 12 inches of brick, is built against the dam, the back of which forms one side of the chamber. The other sides consist of rubble walls 7 feet thick at the base and 3 feet at the top. The chamber is divided into two partitions by means of two walls, projecting 2 feet 10 inches, between which a fish-screen is placed.

Three openings, 30 inches in diameter and located at different heights, serve as the inlet to the gate-chamber. Each opening is provided with a suitable gate. After passing through the screen, the water is drawn from the reservoir by means of a 30-inch cast-iron outlet-pipe, having a stop-cock in the gate-chamber.

The wall was founded entirely on rock, and was built of rubble masonry made of gneiss rock and hydraulic mortar consisting of 1 part of Rosendale cement to 2 parts of sand.

The area of the reservoir is about 60 acres, and its capacity, 240,000,000 gallons.

The original plan was of the Krantz type, as indicated by the dotted line in Plate LVI.; the dam was built, however, in steps, as shown. When the reservoir was first filled, the dam proved to be very pervious, and it has therefore been proposed to build an earthen embankment of 50 feet base at the lowest point of the valley and extending within 10 feet of the overflow against the up-stream face of the wall.

Messrs. Hull and Palmer are the engineers who designed and executed this work.

The Wigwam Dam (Plate LIX.) was built in 1893 to 1896, to form a storage reservoir for the water-supply of the town of Waterbury, Connecticut. The original plans contemplated the construction of a masonry dam 600 feet long having a maximum height of 90 feet above the lowest foundation, and of an auxiliary dam of earth 35 feet high, which was to be built to close a depression. To carry off the flood-water from the watershed of 18 square miles, which supplies the reservoir, a spillway 78 feet long was to be constructed on the north end of the masonry dam, and, in addition to this, a rocky ledge near the earth dam was to be levelled so as to form a second spillway 100 feet long.

As the full capacity of the reservoir is not required now, it was decided to stop the masonry dam, for the present, 15 feet short of its full height and to raise the earth dam only to a height of 20 feet, which makes its top 3 feet higher than the

* Engineering News, April 9, 1887.

crest of the main dam. At this level the rocky ledge mentioned above could not be used as a spillway. A waste-weir, 82 feet long and about 2 feet deep, was made on the north end of the masonry dam by cutting down the rocky hillside and building a wall 5 feet high and 4 feet wide as a continuation of the main dam. This weir is not sufficient to discharge all the flood-water that may reach the reservoir. During a severe freshet in February, 1896, the water rose 6 inches above the top of the masonry dam. No harm was done by this overflow, as the masonry dam has a large margin of safety at its present height.

The body of the dam is built of stone quarried within a mile of the work. The facing consists of fine-grained granite from the Plymouth Quarry, which is situated about 4 miles from the dam. The total amount of masonry laid thus far in the wall has been 14,887 cubic yards. American Portland cement was used for the mortar, except for 5754 cubic yards of the masonry, which was laid in Rosendale cement mortar.

The cost of the dam to date, including engineering, etc., has been $150,000. The present storage capacity of the reservoir is 335,000,000 gallons. This will be increased to 714,000,000 gallons when the dam has been raised 14 feet more.

The facts stated above have been obtained from Mr. R. A. Cairns, City Engineer of Waterbury, who designed the dam and reservoir.

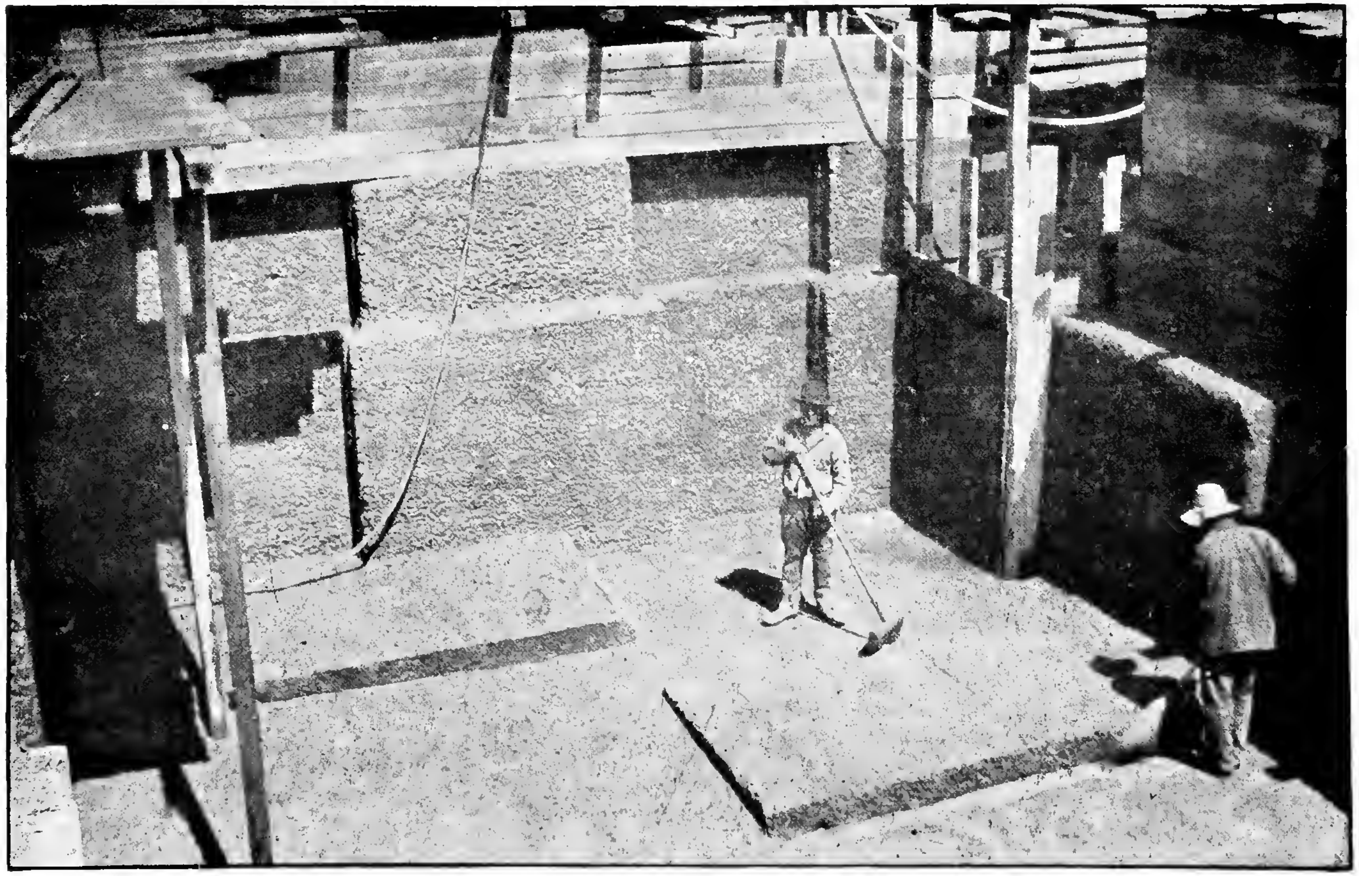
The San Mateo Dam* (Plate LX.) was built in 1887 and 1888 near San Mateo, California, to form a storage reservoir for the water-supply of San Francisco. This reservoir has covered the old Crystal Springs reservoir from which the city was formerly supplied.

The plans originally contemplated building a masonry dam 170 feet high which would store about 31,000,000,000 U. S. gallons. The top-width of the dam was to be 25 feet. At present the dam has only been carried up to a height of 146 feet, as the storage thus obtained is sufficient for the present demand. Its greatest bottom-width is 176 feet. The dam has been curved up-stream to a radius of 637 feet. At the 170-foot level it will have a length of 680 feet.

As no rock suitable for rubble masonry could be found in the vicinity of the work the dam has been entirely built of concrete made with Portland cement mortar, mixed in the following proportions: 22 cubic feet of broken stone, one barrel of Portland cement, and two barrels of sand. The stone used was quarried in small nodules, which were frequently covered with clay and serpentine. It was crushed by machinery and passed through revolving iron cylinders, where it was thoroughly washed by jets of water. All the sand required for the masonry had to be brought from the dunes of North Beach near San Francisco, a distance of about 32 miles. It was first transported in cars a distance of about a mile to barges, towed up the bay for a distance of about 25 miles to a landing opposite San Mateo, and then hauled in wagons to the dam for a distance of 6 miles.

The concrete was mixed in six cubical iron boxes revolved by steam, and was delivered to the work in small cars which were pushed by hand over a double-track tramway. At the dam the tramway was carried on a trestle, built at the top level of the wall, and carried half-way across the valley. The concrete was delivered to

* Eighteenth Annual Report of the U. S. Geological Survey, Part IV.

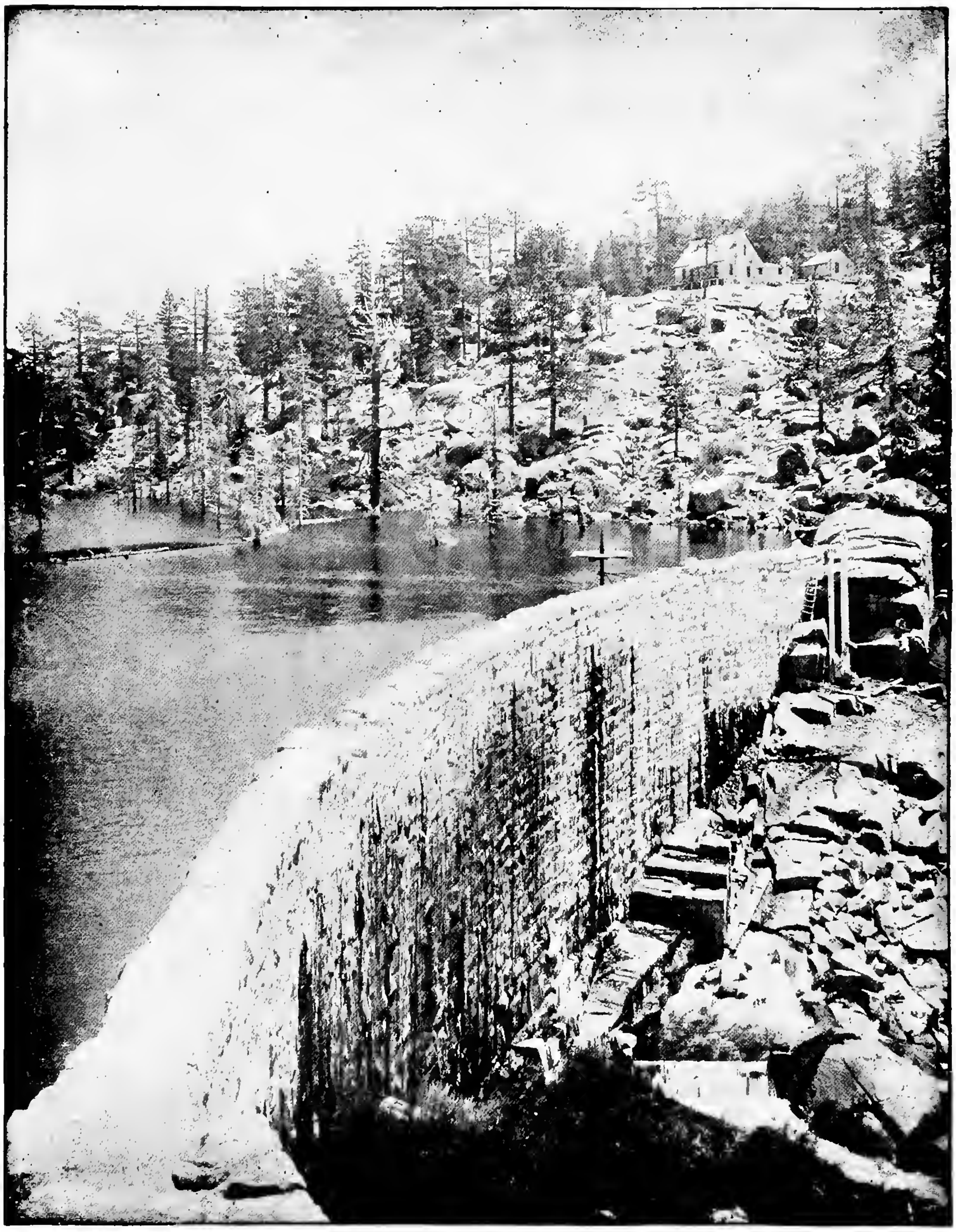


San Mateo Dam.

Roughening Surface of Concrete Blocks to Receive Fresh Cement.

(From "Eighteenth Annual Report of U. S. Geological Survey.")

PLATE E.



BEAR VALLEY DAM.

(From "Eighteenth Annual Report U. S. Geological Survey.")

platforms on the wall through vertical pipes, 16 inches in diameter, which were placed at intervals between the rails of the track. The height from which the concrete was dropped was at times as much as 120 feet, but no injury was done.

The concrete was placed in the dam in large moulds, forming blocks that contained from 200 to 300 cubic feet. These blocks had numerous offsets and were dovetailed together in an ingenious manner. They have been so well bonded in every direction that the dam forms almost a monolith. Since the reservoir has been filled the only signs of any leakage have been a few damp spots in the front face.

The Bear Valley Dam (Plate LXI.) was constructed in 1884 in the Bernardino Mountains in California, to form a large reservoir for irrigation purposes. As all the cement, tools, and supplies had to be hauled for about 70 miles over rough mountain-roads to the site of the dam, and the available financial means were very restricted, the engineer in charge of the work, Mr. F. E. Brown, designed a structure which surpasses in boldness all other dams built. The profile adopted is so thin that the dam cannot resist the thrust of the water by gravity. It owes its stability solely to the curved form of its base, which enables the wall to act as an arch. Assuming the weight of the masonry at 166.7 pounds per cubic foot (corresponding to a specific gravity of $2\frac{2}{3}$) we find that the line of pressure, reservoir full, lies almost entirely outside of the profile.

The work was commenced in the summer of 1883 by the construction of an earthen dam, 6 feet high, about $2\frac{1}{2}$ miles above the site selected for the masonry dam. This dam retained all the water in the stream during the construction, causing it to overflow about 450 acres of land. The masonry dam was built during the latter half of 1884. It was founded on rock and constructed of a rough granite ashlar with a hearting of rubble, all laid in Portland cement mortar or grout. A barrel of this cement delivered at the dam cost $14 to $15, of which amount $10 was for haulage.

The dam is curved up-stream with a radius of 335 feet. Its maximum height is 64 feet. The masonry was carefully laid, the leakage through the dam, when the reservoir was filled, amounting only to a sweating.

The outlet from the reservoir is controlled by a 20 × 24-inch iron sluice-gate which lets the water into a 2 × 3-foot culvert built in the bed-rock. The sluice-gate is operated from the top of the dam by a stem passing through a 6-inch vertical pipe.

The amount of water stored in the reservoir is 1,742,400,000 cubic feet. It is supplied by a watershed of about 56 square miles.

The irrigation company which built the dam intended to replace this rather dangerous structure by a more substantial rock-filled dam, to be built about 200 feet further down-stream. This dam was to have a height of 80 feet. Its foundation was laid in 1893, but nothing more was done in the construction.

The Sweetwater Dam (Plate LXII.)—This dam was constructed in San Diego County, California, by the San Diego Land and Town Company, for storing water for irrigating large tracts of land and for supplying water to National City. The flow of the Sweetwater River, from which water was to be impounded, varies from 1 to 2 cubic feet per second during the dry seasons of the year to 1000 cubic feet per second during periods of freshets.

The construction of the dam and reservoir was decided upon in November, 1886. According to the original plans, the wall was to be formed of concrete and to be 10 feet thick at the base, 3 feet thick at the top, and 50 feet high. On the up-stream side of this concrete dam an earthen bank was to be constructed. After about two months' work had been done Mr. James D. Schuyler, C.E., was given charge of the construction, and wisely modified the plans by deciding to build a substantial dam of rubble masonry, instead of a concrete wall reinforced by an earthen bank.

Owing to the great need of water, the dam was at first carried up to a height of 60 feet, with a profile shown by the dotted lines in Plate LXII. The reservoir thus formed had a storage capacity of 1,221,000,000 gallons. Subsequently the dam was built to a height of 98 feet, increasing the capacity of the reservoir to 5,882,000,000 gallons. The profile adopted is shown by the full lines in Plate LXII.

The principal dimensions of the dam are:

Length at top, ...	380 feet.
Height, ...	90 "
Width at top, ...	12 "
Width at base, ...	46 "

The up-stream face is carried up to within 6 feet of the top of the dam with a batter of 1 in 6. The batter of the down-stream face starts at the base with 1 in 3 for 28 feet, changes then to 1 in 4 for 32 feet, and remains then 1 in 6 to the coping.

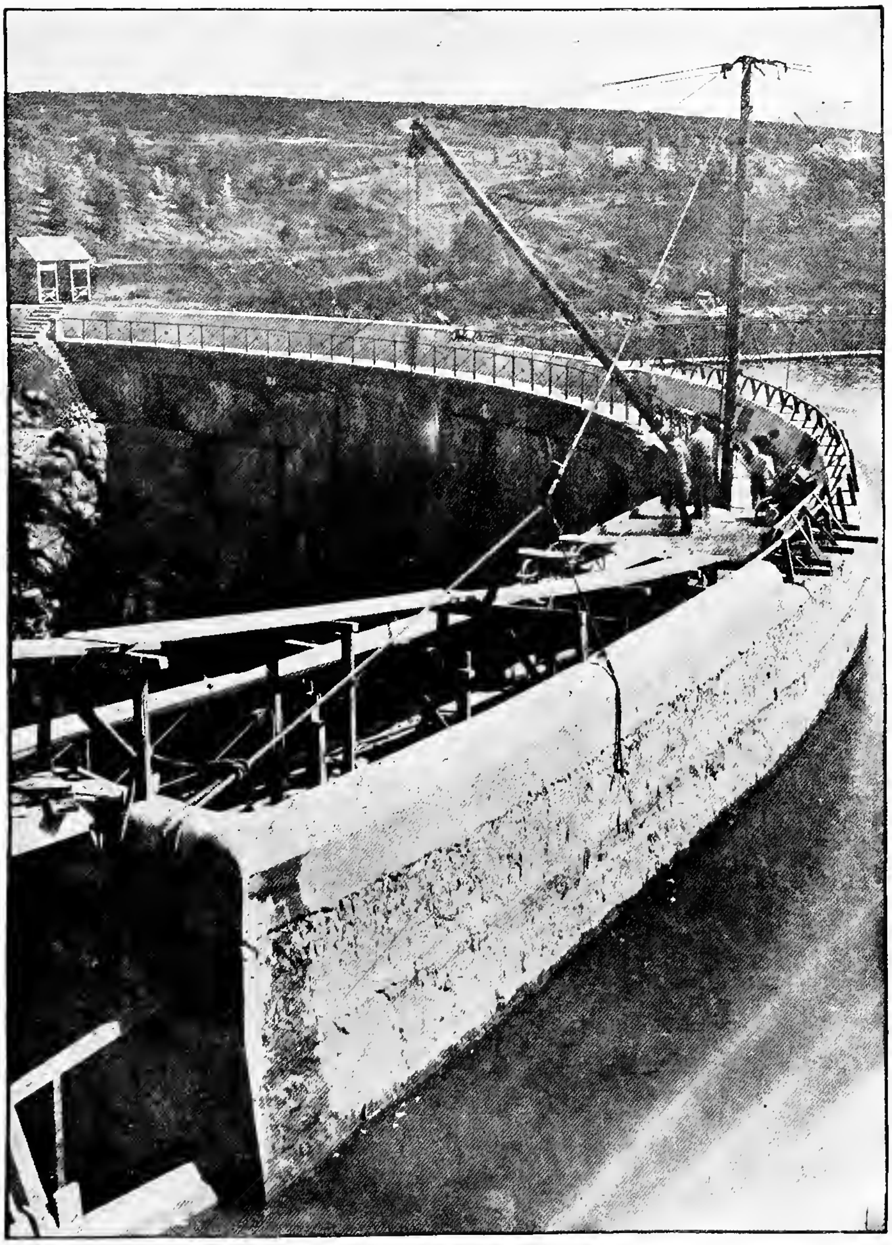
The plan is curved, the radius at the top of the up-stream face being 222 feet. Considerable reliance was placed upon the additional strength obtained by curving the plan, as the line of pressure, reservoir full, would be only one sixth the width of the base from its down-stream toe, if the dam resisted simply by gravity.

The dam was founded on solid rock, which was carefully prepared for the masonry. The stone used was dark blue and gray metamorphic rock, impregnated with iron. It weighed about 175–200 lbs. per cubic foot. The quarry was about 800 feet down-stream from the dam. Portland cement of the best quality was used. It was mixed with clear, sharp river sand in a revolving, square iron box. The usual proportion for the mortar was 1 part of cement to 3 parts of sand; but for the masonry near the up-stream face of the dam only 2 measures of sand were mixed with 1 of cement. The masonry weighed about 164 lbs. per cubic foot. It was all laid by means of four derricks, worked by horse-power.

Water is drawn from the reservoir by means of an inlet-tower, which is located 50 feet up-stream from the dam. It has seven inlet-valves, which are placed at different elevations. Three outlet-pipes, respectively 14″, 18″, and 36″ in diameter, lead from the tower. They have gates on the down-stream side of the dam, by means of which the flow from the reservoir can be regulated.

The waste-weir is formed by part of the dam. It is 40 feet long and 5 feet deep. By means of piers, it is divided into 8 bays. The weir is calculated to discharge 1500 cubic feet of water per second. There is also a 30-inch blow-off pipe, which can discharge 300 cubic feet of water per second.

The Sweetwater Dam was finished April 7, 1888, the construction having required 16 months' time. The amount of masonry laid, including that in the inlet-tower, waste-weir, etc., was 20,507 cubic yards. The average amount of cement used was 1 barrel of cement to 1.17



SWEETWATER DAM.—INCREASING THE HEIGHT OF THE PARAPET.
(From "Eighteenth Annual Report U. S. Geological Survey.")

cubic yards of masonry. The total cost of the work, which was constructed at a time when wages were very high in California, was $234,074, not including the cost of the land.

We have taken the above description from the very complete and interesting paper on the Sweetwater Dam by Mr. James D. Schuyler, the engineer in charge of the work, which paper was read before the American Society of Civil Engineers on October 17, 1888.

Since the above account was written, the dam has been subjected to a very severe test during a flood caused by a rainfall of 6 inches in 24 hours. For 40 hours a sheet of water, 22 inches higher than the top of the parapet, flowed over the dam. The masonry of the dam withstood the strains it had to bear very successfully, not a stone being displaced, but great damage was caused to the outlet-pipes by the erosion of the water below the dam. The repairs required and some changes in the construction of the dam which were deemed advisable cost about $30,000. The alterations made in the dam were as follows:*

1. The parapet of the dam was raised 2 feet and strengthened so as to be able to hold the water permanently level with its crest. For 200 feet, however, the parapet was kept 2 feet lower so as to form a waste-weir, which was provided with iron frames for flashboards, by means of which the waste-weir can be raised to the level of the other part of the parapet.

The effect of this change has been to raise the high-water level in the reservoir 5.5 feet, which adds 25 per cent to the capacity of the reservoir.

2. The original spillway was extended by adding four more bays, each 5 feet wide. All of the bays were carried up to the new crest of the dam.

3. An unused tunnel, 8 × 12 feet in size, which had been excavated to draw down the water in the reservoir during a lawsuit about some of the land required for the reservoir, was utilized as an additional wasteway by placing two 48-inch and two 30-inch pipes in it. These pipes pass through a masonry bulkhead which was built in the tunnel at the reservoir. They are controlled by gate-valves placed in a shaft which reaches the surface.

4. The face of the rock slopes below the waste-channel from the overflow was covered with a grillage of iron rails embedded in concrete.

5. A concrete wall, 15 feet high, was built 50 feet down-stream from the dam and concentric therewith, in order to form a pond 5 to 10 feet deep which acts as a water-cushion for the overflow.

6. The main supply-pipe was replaced and protected through the canyon by means of concrete collars and spur-walls.

The profile of the Sweetwater Dam, while not as slender as those of the Zola and Bear Valley dams, is much bolder than the types now usually adopted. During the flood mentioned above, the line of resistance, though still within the profile, was within a few feet of the outer toe. This must have caused some tension in the masonry at the up-stream face. The safety of the dam has doubtless been due to the excellent manner in which it was built and to the additional strength obtained by building it curved in plan.

The Lagrange Dam[S] † (called also the Turloch Dam) was built in 1890 across the

* Eighteenth Annual Report of the U. S. Geological Survey, Part IV.

† The descriptions marked "S" are taken principally from the report of "Reservoirs for Irrigation," by Mr. James D. Schuyler, published in the Eighteenth Annual Report of the U. S. Geological Survey, Part IV.

Tuolumne River, in California, to form a weir to divert water from the river into two canals which begin at the dam, one on each side of the valley. The dam is 125 feet high. Its top and bottom widths are respectively 24 and 90 feet. The whole dam, which has a length of 320 feet, acts as an overflow-weir. During floods 100,000 cubic feet of water per second pass over the dam at times. As no storage was contemplated, the dam is not provided with pipes. The canyon back of the dam will be allowed to fill with deposit. The dam was built of rubble masonry laid in Portland-cement concrete.

A subsidiary dam, 20 feet high and 120 feet long, was built about 200 feet below the main dam to form a pond 15 feet deep at the main dam, which acts as a water-cushion for the overflow.

The Folsom Dam[8] was built across the American River, in California, to furnish water-power and, also, to divert part of the river to the plains of the Sacramento Valley for irrigation. All of the work was performed by convict labor from one of the State prisons of California.

The dam was constructed at the top of a natural fall in the rock, and is 98 feet high on the down-stream face and only 69.5 feet high at the upper face. It is 87 feet thick at the base and 24 feet at the crest. This dam is about the only one of the structures of this kind erected in the Western States which is not curved in plan. It crosses the river on a straight line and is only curved where it joins the side-wall of the diversion canal. The whole length on top, including the curved part at the canal, is 520 feet. An overflow-weir 6 × 180 feet is formed in the centre of the dam. It can be closed by a single movable shutter consisting of a Pratt truss backed with wood, which is operated by means of hydraulic jacks.

The masonry consists of rough granite ashlar, composed of large blocks weighing 10 tons or more, laid in Portland-cement mortar.

The Hemmet Dam[8] (Plate LXIII.) was built across the south fork of the San Jacinto River, in California, to form a reservoir for irrigation purposes. The enterprise was projected in 1886, but the work on the dam was not begun until January, 1891. According to the original plans the dam was to reach a height of 150 feet above the creek-bed. It was determined, however, to stop the wall, for the present, at an elevation of 122.5 feet, to which height it was brought by the fall of 1895, after various delays caused by freshets. The height above the lowest foundation is 135.5 feet. The dam was built up to an elevation of 110 feet according to the profile designed for a 150-foot dam. At this level, where the dam has a thickness of 30 feet, an offset of 18 feet was made from the front face, and the dam was then carried up 12.5 feet higher, so as to make the top-width 10 feet. Below the 110-foot level the front and back faces are sloped respectively 5 in 10 and 1 in 10. The bottom-width is 100 feet. The lengths of the wall on top and at the bottom of the valley are respectively 280 and 40 feet. The dam is curved in plan to a radius of 225.4 feet. A notch, 1 × 50 feet, was left in the wall to act as a waste-weir, but during severe freshets the water overflows the whole dam.

The dam contains 31,105 cubic yards of granite rubble masonry. The large stones were placed at least 6 inches apart, the spaces between them being filled with concrete made with Portland cement mortar in the following proportions, viz.: 1 part cement,

3 parts sand, and 6 parts stone crushed to pass through a 2½-inch ring. The mortar and concrete were mixed in iron boxes revolved by water-power. All the cement used in the dam (about 20,000 barrels) had to be hauled for 23 miles up the mountain to an elevation of 500 to 600 feet, over grades of about 18 per cent. The cost of a barrel of cement delivered on the ground was about $5.00.

Two 22-inch pipes (respectively at the 45- and 75-foot levels) form the outlet from the reservoir. Their up-stream ends are turned upwards by elbows and flared to 30 inches in diameter. The pipes can be closed in the reservoir by hemispherical covers operated by wire ropes, each passing over a pulley and windlass on top of the dam, but the covers are usually raised and replaced by fish-screens, the outlet-pipes being controlled by stop-cocks set below the dam.

The Colorado Dam* (Plate LXIV.) was constructed in 1892 across the Colorado River, about two miles above Austin, to furnish power for pumping that city's water-supply, for electric lighting, for propelling street cars, and for general manufacturing purposes.

At the site selected for the dam, the river flows in a deep gorge in limestone, with bluffs on either side rising as high as 150 feet. The dam was founded on the rock forming the river-bed, which was only excavated at the faces of the wall, to a depth of about 4 feet. It was constructed entirely of masonry, the faces and the coping being formed of blue granite that was quarried in Bennet County, Texas, at a distance of 80 miles from the work, the balance of the dam being built of rubble masonry, composed of hard limestone, obtained at the site of the dam, and of hydraulic mortar composed of 1 part Portland cement to 3 parts of sand. The coping stones were fastened by iron dowels and clamps.

The masonry laid in the dam amounted to about 18,000 cubic yards of granite cut stone and about 70,000 cubic yards of limestone rubble, the price paid for the former class of masonry being $11—$15, and for the latter $3.60 per cubic yard. Fifty cents additional price per cubic yard was paid where Portland cement was used.

Including the bulkheads, at either side, the length of the dam is 1275 feet, of which 1125 feet form the overflow-weir.

The water-shed above the dam contains about 50,000 square miles, from which a maximum quantity of water of about 250,000 cubic feet per second flows over the dam.

The lake formed by the dam is 25 miles long. A canal 90 feet wide and 15 feet deep conveys the water to the turbine-wheels.

The power obtained is estimated at 14,636 H.P. for 60 working hours per week, of which 720 H.P. are required for pumping the city's water-supply.

The cost of the dam was about $570,000, and the cost of the entire work, including dam, power-house, reservoir and distributing system, was about $1,400,000.

The whole cost was borne by the city of Austin. The works were designed and constructed under Mr. Joseph Frizzell, Chief Engineer, and Mr. John Bogart and Mr. J. T. Fanning, Consulting Engineers.

The contractor was Mr. Bernard Corrigan of Kansas City, Mo.

The Dam of the Butte City Water Company† was constructed in 1892 to form a reservoir for the water-supply of Butte City, Mont.

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* See Engineering News, July 11, 1891. See Report on the Austin (Colorado) Dam by J. T. Fanning, Consulting Engineer, June 22, 1892.

† Engineering News, December 15, 1892.

The dam is located about 5900 feet above the level of the sea in a region where there is practically no rain, the reservoir being filled by melting snow. Its principal dimensions are:

Top-width, ...	10 feet.
Bottom-width, ...	83 "
Maximum height, ...	120 "
Length on top, ...	350 "
Radius of plan, ...	350 "
Length of waste-weir, ...	15 "

The reservoir has an area of 130 acres and a capacity of 1,000,000 U. S. gallons.

The dam was constructed of concrete (made of crushed granite and Yankton Portland-cement mortar), faced with hard blue granite.

A 20-inch waste-pipe and two 20-inch supply-pipes pass through the masonry.

The water is conveyed to the city by a banded, red-wood stave pipe 24 inches in diameter, 9 miles long, and by a lap-welded 20-inch steel pipe 3 miles long.

The works were designed by Mr. Chester B. Davis, the Chief Engineer of the Water Company.

The Sodom Dam (Plate LXV) was constructed in 1888–1893 to form a new storage reservoir for the water-supply of the city of New York. The work was performed under the direction of the Aqueduct Commissioners (who were given charge of the construction of the New Croton Works by Chapter 490 of the Laws of 1883), Mr. A. Fteley being Chief Engineer.

The Sodom Reservoir and the Bog Brook Reservoir form together what is known as the "Double Reservoir I" on the East Branch of the Croton River. While these two basins have about equal storage capacities, the water-shed of the former is about twenty times as large as that of the latter, the areas of the water-sheds being respectively 73.42 and 3.5 square miles. To compensate for this difference, the two basins are united by a tunnel 10 feet in diameter and 2000 feet long.

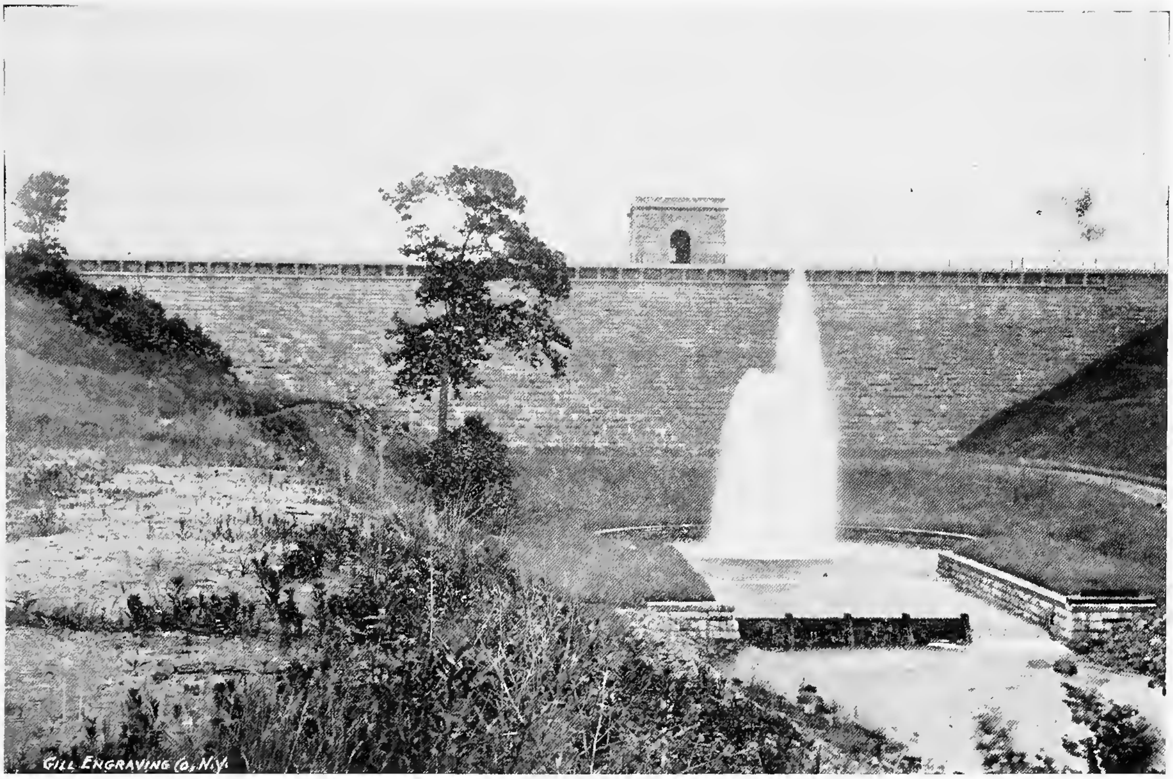
The storage capacity of the double reservoir I is about 9,500,000,000 gallons.

The Sodom Reservoir is formed by a masonry dam, built across the East Branch of the Croton River, and by an earthen bank about 9 feet high and 600 feet long, constructed nearly at right angles to the masonry structure on a ridge to the east of it. The earthen dam is continued by a masonry overflow-wall about 8 feet high and 500 feet long, its top being at an elevation of about 415 feet above mean tide in the Hudson River at Sing Sing.

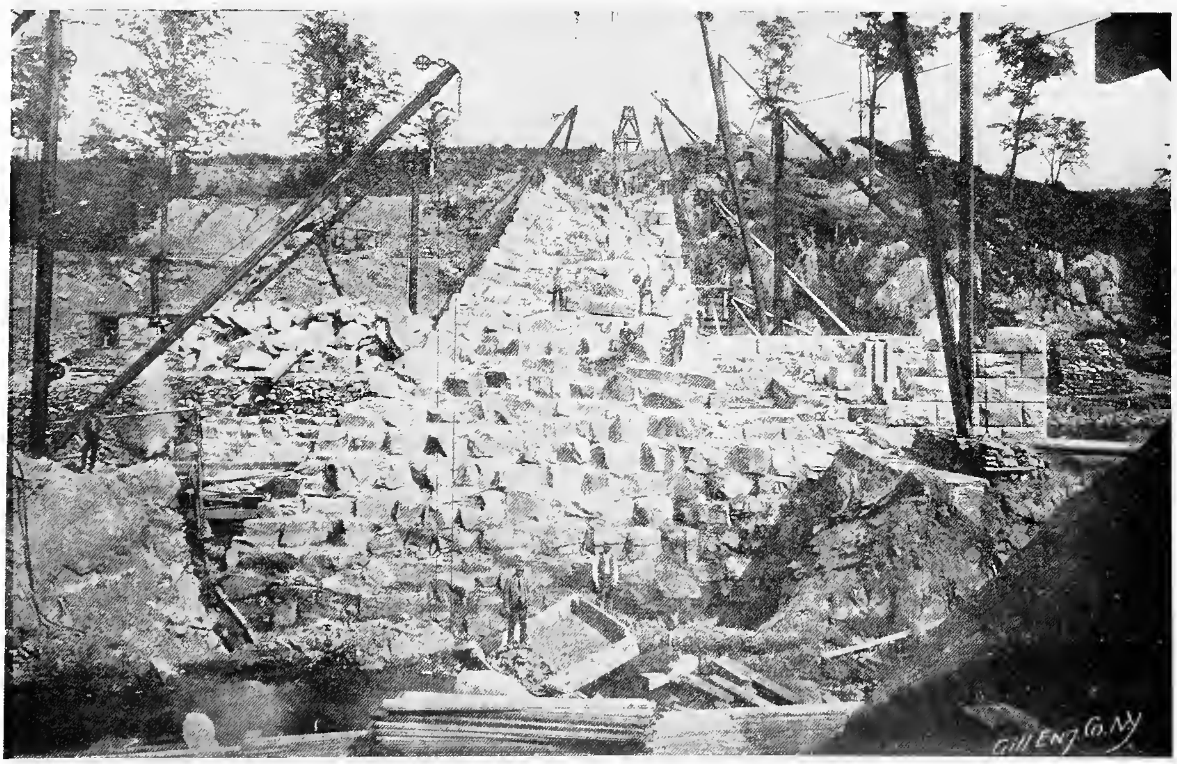
The principal dimensions of the masonry dam are as follows:

Length at coping, ...	500 feet.
Maximum height above foundation, ...	98 "
" " " ground, ...	78 "
Top-width, ...	12 "
Width at foundation, ...	53 "

PLATE G.



SODOM DAM.



SODOM DAM, IN CONSTRUCTION.

The total amount of masonry placed in the structure was 35,887 cubic yards.

Near the centre of the dam a gate-house, 37 feet by 42 feet, was built for controlling the flow from the reservoir, which takes place through two 48-inch cast-iron pipes.

The masonry was laid with the utmost care. The foundation, which was throughout solid rock, was swept with wire stable-brooms and washed clean by means of streams from hose-pipes. The irregularities of the bed-rock were generally levelled by layers of concrete made with Portland cement. Where water issued from cracks in the rocks, however, better results were obtained by laying rubble made of small stones, by which the water was confined to small wells about 2 feet in diameter. When the mortar of the rubble masonry had set sufficiently, the wells were bailed out and quickly filled with dry mortar into which large rubble stones were bedded.

The mortar consisted principally of Portland cement and sand, mixed 1 to 2 in the lower and upper parts of the wall and 1 to 3 in the middle part.

An interesting feature of the construction of the Sodom Dam was the use of a steel cable, 2 inches in diameter and weighing 7 lbs. per foot, which was stretched across the valley and served for delivering the building materials on the wall. The cable was stretched across two towers, 667 feet apart, and anchored into the bed-rock.

For a full description of the details of the construction of the Sodom Dam we refer the reader to a paper on this subject written by Mr. Walter McCulloh, M. Am. Soc. C. E. and published in the transactions of the American Society of Civil Engineers for March 1893.

Owing to the great care taken in laying the masonry in the Sodom Dam, this structure has proved to be perfectly water-tight. In this connection we quote the following remarks from the paper just mentioned:

"As to the water-tightness of Sodom Dam, it is perfect. When the reservoir is filled (with 68 feet of water behind the wall) many careful examinations have failed to disclose any leaks whatever, either through the wall or under it, or through the rock around the ends in the side hill. 'Sweating' at the joints in the facing stone appears at several points only, but not in sufficient quantity to produce a trickle. What moisture there is will wholly disappear on a dry, clear day; but if the day be humid, dampness is visible upon the face of the stone as well as at the joints."

The contract for the Sodom Dam and its appurtenances was awarded to Sullivan, Rider & Dougherty on December 30, 1887. Ground was broken on February 22, 1888. Owing to various delays the work was not finished and finally accepted by the Aqueduct Commissioners until October 31, 1892.

The engineers in immediate charge of the work under the directions of the Chief Engineer were Mr. George B. Burbank, Division Engineer, and Mr. Walter McCulloh, Assistant Engineer. On the resignation of the former, June 17, 1891, the latter was appointed Division Engineer and had charge of the work to its completion.

The Titicus Dam (Plates LXVI. to LXXII.) was constructed in 1890 to 1895 across the Titicus River, an affluent of the Croton, near the village of Purdy's Station, N. Y., to form a storage reservoir for the water-supply of the city of New York.

The dam consists of a central wall of masonry which is extended on each side

by an earthen dam. The central masonry structure, part of which forms the overflow-weir, has a length of 534 feet. The lengths of the north and south earthen dams are respectively 732 and 253 feet, the whole length of the dam being 1519 feet.

The masonry portion of the dam was founded entirely on rock. The earthen dams were provided with masonry core-walls which were founded on hard-pan with the exception of a short distance on both sides of the masonry dam, where a rock foundation was obtained.

The principal dimensions of the masonry dam are:

Width under coping, ...	20.7 feet.
Width about 109 feet below coping, ...	75.2 "
Maximum height above foundation, ...	135.0 "
Maximum height above surface, ...	109.0 "

The waste-weir or overfall, which has a length of 200 feet, is built according to the stepped profile shown on Plate LXX.

The masonry consists of rubble, faced up-stream and down-stream with cut stone, laid in regular courses. The bulk of the rubble is composed of large stones, containing 3 to 30 cubic feet, the spaces between them being filled with mortar, into which small stones are bedded.

The cornice of the dam, the top of the overflow, and the superstructure of the gate-house are constructed of granite dimension-stone. All the stone required for the dam was obtained from a quarry situated about 1½ miles from the work. It was transported on a tramway, partly by gravity and partly by means of a small locomotive.

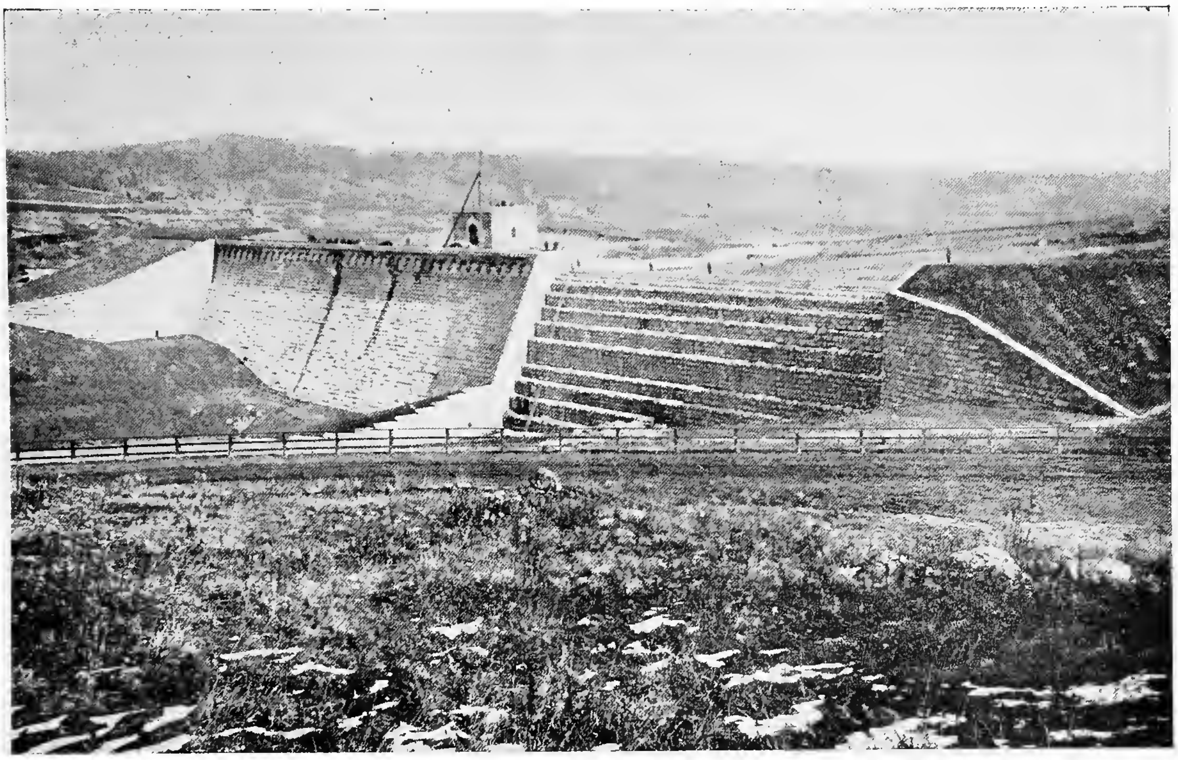
Both American and Portland cement were used for the mortar, which was usually composed of 1 part cement to 2 parts of sand. Part of the masonry was laid during freezing weather, the mortar being mixed with brine and the sand heated. The stones were steamed before being laid. No masonry was laid, however, when the temperature was below 20° Fahrenheit.

Thirty-six masons with six derricks were usually employed on the wall. They laid on an average 3240 cubic yards of masonry per month and a maximum of 5700 cubic yards.

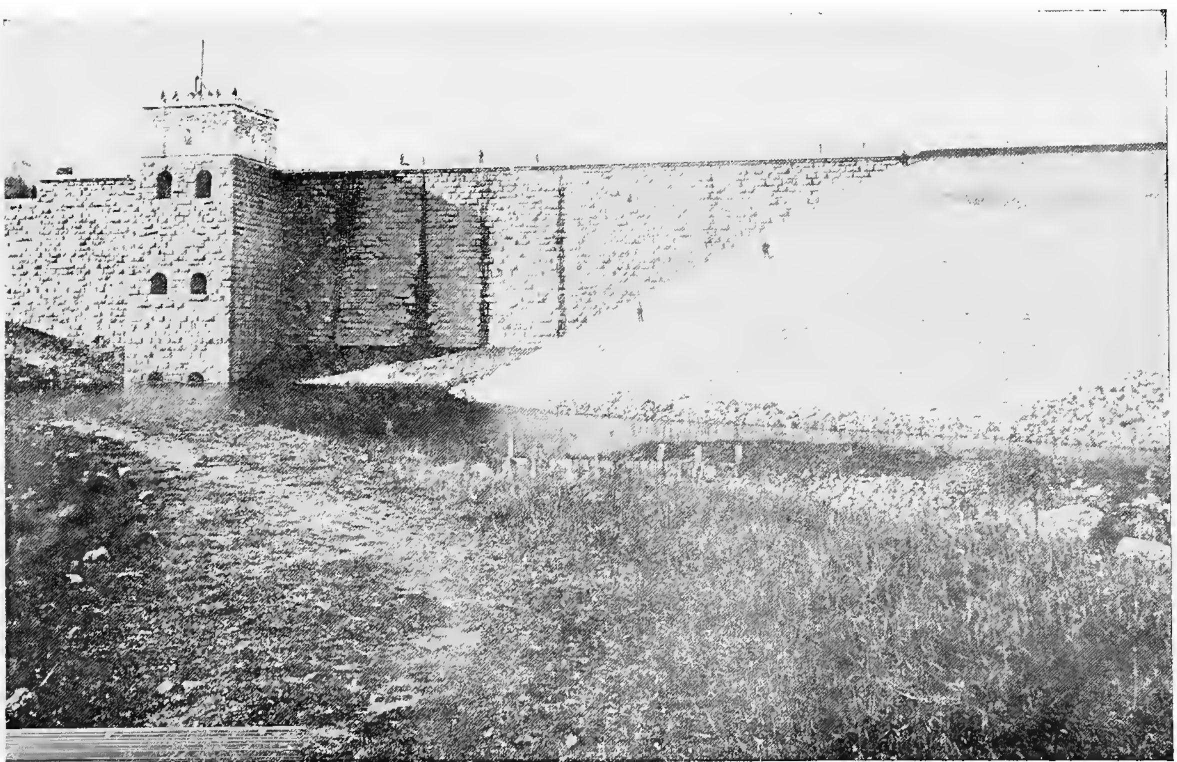
The earthen dam, constructed on both sides of the masonry structure, has a maximum height of 102 feet above the surface and rises 9 feet above the crest of the overflow-weir. It has a top-width of 30 feet and slopes of about 2½ to 1. The up-stream face is covered with a paving of stones (18 inches deep, laid on 12 inches of broken stone) which extends 5 feet above the top of the overflow. The top of the dam, the down-stream slope, and the up-stream slope above the paving are sodded.

The core-wall, which is constructed of rubble masonry, is 5 feet wide on top, and 17 feet wide at a depth of 98 feet, both faces being battered about .06 foot per foot. Below this depth both faces are vertical. The core-wall has a maximum height of 124 feet above the foundation.

The flow from the reservoir is regulated by a gate-house, which is constructed



TITICUS DAM, FRONT FACE.



TITICUS DAM, BACK FACE.

on the up-stream face of the dam, near the overflow-weir. A central wall divides the substructure of the gate-house into two divisions, each of which is divided by a cross-wall into an inlet and an outlet water-chamber. The former has three inlet openings (6 feet wide and 8 to $9\frac{1}{2}$ feet high): one at the surface of the reservoir, one at mid-depth, and one at the bottom. These openings are protected by screens made of $\frac{1}{2} \times 2\frac{1}{2}$-inch iron. They can be closed by means of stop-planks or wooden drop-gates which are placed in grooves provided in the side-walls of the substructure. There are two sets of grooves, 2 feet 5 inches apart. By placing stop-planks in them and filling the intervening space with a puddle of clay and earth a tight coffer-dam can be built which cuts off the gate-house securely from the reservoir. In ordinary cases one set of stop-planks suffices for this purpose, if the joints are properly calked.

The cross-wall between each inlet and outlet chamber has two openings (one at the bottom and one at mid-depth) which are controlled by 2×5-foot sluice-gates, operated from the floor of the gate-house. The top of the cross-wall forms an overflow-weir, the height of which can be regulated by means of stop-planks. Two sets of grooves are provided in the side-walls for these stop-planks, as at the inlet openings.

Two 48-inch outlet-pipes (one for each division of the gate-house) convey the water from the outlet-chambers to the old channel of the Titicus River, which was excavated to rock for a short distance. Each of the lines of outlet-pipes is controlled by a stop-cock placed in a vault about 80 feet below the gate-house. Besides these pipes, a 24-inch drainage-pipe, that was used during the construction of the reservoir, passes through the dam. Its up-stream end is closed by a flap-valve.

The superstructure of the gate-house is 32 feet 6 inches $\times$ 35 feet in plan. It is constructed of granite and has a roof of brick arches sprung from I beams. The floor of the building consists of a cast-iron grating supported by I beams.

Before the work on the dam was commenced, the Titicus River was diverted by building a crib-dam about 1000 feet above the site of the masonry dam. A new channel 1000 feet long (25 feet wide and 8 feet deep) was excavated on the south bank of the river. It was continued by a wooden flume (Plate LXXI.), about 600 feet long, which passed through the masonry dam about 25 feet above the original bed of the river. The flume had two compartments, each 9 feet wide by 7 feet 9 inches high. After the masonry dam had been brought up to the top of the flume, the latter was turned so as to discharge the water it carried into the lowest inlet of the gate-house, whence the water escaped through the 48-inch outlet-pipes. Ordinarily these pipes could discharge all the water flowing in the Titicus River. Provision was made for freshets by keeping part of the overflow-weir 10 to 15 feet below the rest of the masonry. During floods the valley above the dam would fill with water until the depression left in the overflow-weir was reached, where the water would be discharged.

The plans for the work were made by Mr. A. Fteley, Chief Engineer of the Aqueduct Commission of the City of New York. Mr. Charles S. Gowen and later on Mr. Alfred Craven had charge of the work as Division Engineer. Mr. Robert Ridgway was in immediate charge of the work as Assistant Engineer.

The contract for the Titicus Reservoir was let on February 18, 1890, to Washburn,

Shaler & Washburn, who constructed the work in an excellent manner. The reservoir was practically completed by January 1, 1895.

The Old Croton Dam (Plate LXXII.) was constructed in 1837 to 1842 across the Croton River, to form a storage reservoir for the City of New York. The Old Croton Aqueduct, which has a length of about 41 miles, begins at this reservoir.

At the site selected for this dam the river-channel was 120 feet wide, the ordinary depth of the water being about 4 feet. During freshets this depth was increased to a maximum of about 10 feet. The left bank of the river consists of abrupt rocks, while the right bank is formed of a sandy table-land, about 3 feet higher than the ordinary level of the river, extending back 80 feet to a hill of sand, having a slope of about 45°.

On the location selected for the dam a rock foundation could only be obtained near the south bank. It was, therefore, determined to form the dam of earth, with the exception of the overflow-weir, which was to be constructed of masonry and to be located at the southern extremity of the dam. On the down-stream slope of the earthen bank a protection-wall was to be built.

According to the original plans the overflow-weir was to be 100 feet long, and to be flanked by abutments rising 8 feet above its crest, but, owing to the short distance that the rock extended into the river, the length of the weir was reduced to an average of 90 feet, part of it being obtained by excavating the rock on the south bank. Only the north abutment had to be constructed, the one on the south being formed by the rock. As the length of the weir had been reduced, the north abutment was raised so as to be 15 feet above the crest of the weir on the up-stream, and 12 feet on the down-stream, side. The rock descended so rapidly in the river that an artificial foundation had to be prepared for part of the north abutment.

A waste-culvert, 5 feet by 6 feet, having two sets of suitable gates, operated from a small house on top, was constructed in the abutment, to make it possible to draw down the reservoir whenever it should be required, for repairs or other purposes. A small foot-bridge, placed across the waste-weir, gives access to the gate-house.

Before the earthen dam had been quite completed it was washed away by an unprecedented freshet which occurred during the night of January 7–8, 1841. The gap made by the destruction of the earthen dam was about 200 feet wide. It was decided to fill it up by extending the masonry overflow-weir 180 feet across the channel of the river to a point where it would join the earthen dam near the north bank. As no rock foundation could be obtained for the extension of the masonry structure, an artificial one had to be prepared. The method adopted was as follows (see Plate LXXII.):

The bottom of the river was cleared of mud and boulders where the masonry was to be built. The space to be occupied by the structure was then enclosed by coffer-dams, formed of heavy cribs which were left in the foundations. The cribs *C* and *D* were first sunk, and covered on top by white-pine planks, 6 inches deep. On top of these cribs two others were placed, and connected together near the top by cross-ties. While the cribs were being carried up, the space between them, *E*, was filled with concrete. In front of *D*, a small crib, *H*, having square timbers only on

its down-stream face, was constructed and securely anchored by timber ties to *D*. The cribs just described formed a coffer-dam on the up-stream side of the foundation. As a protection against the water on the down-stream side the cribs *J*, *K*, and *L* were placed, the crib *J* being filled with concrete and the others with loose stones. On top of these cribs an apron of elm timber was constructed. The square timbers in all the cribs were 12 × 12-inch hemlock. The cross-ties were of oak and were spaced 6 to 10 feet apart. They were dovetailed into the square timbers, and secured by treenails 1 inch in diameter. The crib timbers were fastened together by treenails 2 to 2½ inches in diameter by 30 inches long, placed about 3 feet apart. The planking of the apron was secured to the square timbers on which it rested by 1-inch locust treenails.

After the coffer-dams had been completed the space enclosed by them was excavated to a hard-pan foundation and filled with concrete and masonry as shown on Plate LXXII. Against the up-stream face of the dam an earthen bank, having a slope of 5 in 1, and extending on the bottom to a width of 275 feet, was constructed. Near the top, the earth bank was paved with stone. Three hundred feet down-stream from the Croton Dam, a secondary dam was constructed of cribs of round timber, filled with stone, the object being to back up the water so as to form a pool to break the force of the water flowing over the weir of the main dam, and, also, to keep the cribs and apron constantly under water.

Since 1842, when the first water-works were completed, the supply of the city of New York has depended on the stability of the Old Croton Dam. As the crest of the masonry part of the dam forms an overflow-weir of only 180 feet in length for a watershed of 360 square miles, much apprehension has been felt, at times, about the safety of the old dam. It has, however, thus far stood, successfully passing during severe freshets a sheet of water over 8 feet deep. Within the next ten years the old dam will be replaced by the New Croton Dam, a masonry wall 290 feet high, which was begun in 1892. This dam is located 3 miles below the old one, which will be submerged about 30 feet when the new reservoir is formed. It was originally intended to build the new dam about a mile further down-stream at the old Quaker Bridge. This project was long before the public. It was finally decided to substitute for it the construction of a dam about a mile further up-stream, which is now being built and is known as the New Croton Dam. As the profile of this dam is based entirely upon that designed for the Quaker Bridge Dam, the exhaustive studies made for the latter have lost none of their interest.

The Quaker Bridge Dam (Plate LXXIII.)*—The plans for the new Croton Aqueduct, which was built in 1884–1891 for the city of New York, included the construction of an immense storage reservoir whose capacity was to be 32,000,000,000 gallons. This artificial lake was to have a surface containing about 3900 acres, and was to be supplied by a watershed of 361 square miles. It was to be formed by closing the Croton Valley about 4½ miles below the present reservoir by a gigantic masonry dam, 1350 feet long, and about 270 feet high at the deepest part of the valley. This structure was named after a bridge near the proposed site, the Quaker Bridge Dam.

* See New Croton Dam, page 105.

According to Chapter 490 of the Laws of 1883, the Department of Public Works of the city of New York is required to prepare all the plans for the new aqueduct and reservoirs, but the construction of these works is entrusted to a commission composed of city officials and private citizens. The first design for the Quaker Bridge Dam was prepared, therefore, under the direction of Mr. Isaac Newton, Chief Engineer of the Department of Public Works, who was assisted by E. S. Chesbrough, J. W. Adams and J. B. Francis, as consulting engineers. After receiving this plan, the Aqueduct Commissioners, who were fully impressed with the magnitude and importance of the proposed work, ordered their own Engineer Department, at the head of which they had placed Mr. B. S. Church as Chief Engineer, Mr. Alphonse Fteley as Deputy Chief Engineer, and Mr. J. P. Davis as Consulting Engineer, to make a new and thorough research on the subject of masonry dams. Mr. Fteley was given special charge of this investigation. The mathematical part of the studies was assigned to the writer, who was assisted in this work later on by Mr. Ira A. Shaler.

After protracted investigations the profile shown in Plate LXXIII. was finally presented to the Aqueduct Commission. Although this design was not then finally accepted, it has been long before the public, and we think, therefore, that a description of it, taken from the published reports of the Chief Engineer, Mr. B. S. Church, and of the Consulting Engineer, Mr. Fteley*, may be of some interest.

The profile was based upon the following data and conditions:

	Elevation, in feet, referred to Croton Datum.†	Bed-rock.
Top of dam,	210	262
Highest water-level,	206	258
River-bed,	35	87
Bed-rock,	−52	0
Base of dam,	−58	−6

The top width of the dam is to be 20 feet.

The water-pressure is supposed to act on the back face of the dam from elevation 206 to $-52 = 258$ feet; and on the front face from elevation 35 to $-52 = 87$ feet. Only the horizontal thrust of the water is considered, the vertical component being neglected. The small error resulting from this omission is in the direction of safety.

The wind-pressure is not considered in the calculations.

The weight of the masonry is assumed to be $156\frac{25}{100}$ pounds per cubic foot (corresponding to a specific gravity of 2.5); this weight being determined by experimental blocks.

The weight of the water is taken as 62.5 pounds per cubic foot.

The weight of the gravel saturated with water, which lies on the slopes of the dam below the river-bed, is assumed to be 145.88 pounds per cubic foot, which equals 94 per cent of an equivalent volume of masonry. This figure is determined by taking the gravel

* Mr. Fteley resigned the position of Deputy Chief Engineer on July 31st, 1886. He acted as Consulting Engineer of the Aqueduct Commission until November 1888, when he succeeded Mr. Church as Chief Engineer.

† Croton Datum is the average mean tide at Sing Sing, about 30 miles above New York.

to weigh 125 pounds per cubic foot, and by supposing one third of its bulk to be filled with water.

In all the calculations one cubic foot of masonry is taken as the unit of weight.

The masonry is assumed to be impervious to water.

The profile is to comply with the following conditions:

First. The lines of pressure are to lie within the centre third of the profile, whether the reservoir be full or empty.

Second. The pressures in the masonry are not to exceed the following limits: For a depth of water of 110 feet or less, 8.2 tons of 2000 lbs. per square foot at the front face, and 10.3 tons of 2000 lbs. per square foot at the back face (these limits being equal to 8 and 10 kilos. per square centimetre respectively). From a depth of 110 feet to the base of the dam the pressures are to increase gradually so as to reach a maximum amount of 15 tons of 2000 lbs. per square foot at the base. The pressures in the masonry are to be calculated by formula A or B, page 10.

Third. The dam is to have ample safety against shearing or sliding.

The conditions stated above differ from those given by Rankine and other recent authorities only in the high limit of pressure adopted for the lower part of the dam. This departure from the usual recommendations was rendered necessary by the great height of the dam. Had the limits of pressure of 8.2 tons per square foot at the front face and 10.3 tons at the back face been used for the whole dam, the width of the base would have been about 350 feet, and the faces at the base would have become exceedingly flat. Under these circumstances it cannot be supposed that the maxima pressures at the base would occur at the faces in accordance with formulæ A and B. The thin triangles of masonry between the faces and the base could not transmit great pressures, and would therefore involve a waste of material. Some practical limit must evidently be placed to the flatness of the faces of a dam. This consideration resulted in the profile for the Quaker Bridge Dam being designed with the pressures in the masonry increasing gradually towards the base, where a maximum strain of 30,000 lbs. per square foot would be reached in the *Theoretical Profile*. The necessity of confining the lines of pressure within the centre third of the profile precluded the use of such high pressures in the upper part of the dam.

Although the maxima pressures in the Quaker Bridge Dam will be considerably above the limits usually adopted for similar structures, yet they will exceed but slightly the pressure of about 28,660 lbs. per square foot sustained by the masonry of the Almanza Dam successfully for three centuries. The materials to be employed in the wall will be sufficiently strong to resist much greater stresses than those to which they will be subjected.

The theoretical profile for the Quaker Bridge Dam was calculated by the method* we have explained in Chapter III., Equations (1) to (7) being used with the following modifications: Equation (1), page 17, is based upon the assuption that the water-surface

* For the preliminary profiles the writer devised a method of trial-calculations, which consists in estimating the probable length of a given joint from that of the joint above, the correctness of the assumed length being tested by taking moments about a vertical axis as explained in the method given for checking a profile, in Chapter III., page 23. This simple but laborious process was subsequently improved by substituting the exact equations given in Chapter III. for the trial-calculations.

is at the top of the dam. As the highest flow-line at the Quaker Bridge Dam was assumed to be 4 feet below the top of the dam, the depth to which both faces could be continued vertical was found by the following formula:

Let b = the superelevation of the dam above the highest water-level. Using the letters given on page 14, we have

$$x = a = u + v + n. \qquad (1)$$

Substituting

$$u = \frac{a}{3}, \quad n = \frac{a}{2}, \quad v = \frac{M}{W} = \frac{\frac{d^3}{6r}}{(d+b)a},$$

we find, by reducing,

$$d^3 - ra^2d - ba^2r = 0.$$

The next courses of the dam were determined by using Equations (2), (3), (4) and (6) to elevation 96, which is at a depth of 110 feet below the surface of the water. It was decided to give one batter to each face of the dam from this elevation to the base, where the limiting pressure should equal 15 tons of 2000 lbs., both for reservoir full and empty. The gravel resting on the dam below the river-bed complicated the conditions. Mr. Ira A. Shaler, Assistant Engineer, found the following equations for determining the width of the base by making the proper substitutions for this special case in the general Equations (I) and (III), pages 16 and 20:

$$x^2[p + q - h(1 + \beta\vartheta^2)] + x[lh(\beta\vartheta^2 - 1) - 2w] = 6M;$$

$$y[hx(1 + 2\vartheta^3\beta - 3\vartheta^2\beta) + hl(2 - 2\vartheta^3\beta) + 6w]$$
$$= hx^2(1 - \vartheta^2\beta + \vartheta^3\beta) - hl^2(1 - \vartheta^3\beta)$$
$$+ hxl(1 - 2\vartheta^3\beta + \vartheta^2\beta) - 6wm$$
$$+ 4wx - qx^2;$$

in which

β = the ratio of unit weight of gravel to unit weight of masonry;

$\vartheta = \frac{g}{h}$, g being the depth of gravel overlying the base.

The other letters are used as on page 14.

Having calculated the theoretical profile, it was modified in the following manner in order to obtain a practical design:

1st. A few simple batters were substituted for the many changes in the theoretical form.

2d. The thickness of the profile was increased slightly between the top of the dam and elevation 170 (between which limits the water-surface is supposed to fluctuate) in order to increase the strength of this part of the wall against shocks from floating bodies, and also to add to the symmetry of the profile.

Finally, a few steps were substituted for the sharp triangle of masonry between the

front face and the base. This change reduced the width of the base from 230 feet to 216 feet, avoiding thus a considerable amount of expensive excavation.

The practical profile being designed, the pressures in the masonry were calculated, and the following results obtained:

Maximum pressure at front face,	. .	15.4 tons of 2000 lbs. per sq. ft.
" " at back face,	. .	16.6 " " " "
Average " on base,	. . . .	10.5 " " " "

As regards the plan of the dam, the question whether it ought to be curved or straight was discussed fully in the reports of the Chief Engineer and Consulting Engineer. Both these gentlemen recommended that a straight plan should be adopted on account of the great width of the valley.

In concluding our description of the proposed Quaker Bridge Dam, we wish to state that, while this structure will be about one hundred feet higher than any existing dam, the pressures at its base are within limits that the materials to be employed in the construction fully warrant, and exceed but slightly those sustained safely in the Almanza Dam for more than three centuries. The profile for the Quaker Bridge Dam has been based upon principles which the experience with many high masonry dams, built within recent years abroad, has proved to be safe, and no apprehensions need therefore be felt as regards the strength of the proposed dam to withstand successfully the thrust of the water in the reservoir and the crushing strains in its masonry.

[*Note.*—Since the above description of the proposed Quaker Bridge Dam has been written, the Aqueduct Commissioners have appointed Joseph P. Davis, James J. R. Croes, and William F. Shunk as a Board of Experts to consider the plans proposed for this dam. The following extracts from the report of these eminent engineers give the conclusions at which they have arrived as regards the profile and plan of the dam:

EXTRACTS FROM REPORT OF THE BOARD OF EXPERTS.

NEW YORK, October 1, 1888.

To the Honorable the Aqueduct Commissioners:

By a resolution of the Aqueduct Commissioners, adopted March 7th last, and by subsequent action, the undersigned were appointed a Board of Experts to take into consideration the plans of the Quaker Bridge Dam, as projected by the Engineers of the Commissioners, and modifications which had been or might be suggested by others, either in plan or cross-section, and to fully advise the Commissioners on the subject.

We have found that the work assigned to us required much more extended investigations than were anticipated, but we have at length finished them, and now have the honor to report the conclusions at which we have arrived.

The proposed location of the Quaker Bridge Dam is at a point on the Croton River, at about two miles above its mouth, where the steep sides of the valley approach to form a ravine. This ravine is about 1300 feet wide at an elevation of 230 feet above tide level, 300 feet wide at the level of the river-bed, 35 feet above tide, and has a rock bottom 87 feet below the stream level, or 52 feet below the tide level in the Hudson River.

It is proposed to close this ravine with a masonry dam which will impound the

waters of the stream and raise the water level to a height of 200 feet above mean high tide. The greatest height of the dam from foundation level to the top of road parapet will be, therefore, from 265 to 270 feet, depending upon the character of the surface of the rock at the deepest point.

It will be about 100 feet higher than any dam yet built.

It is to impound upwards of 5,000,000,000 cubic feet of water in an artificial lake 16 miles long and 165 feet deep at its lower end.

The water-shed tributary to it has an area of 361 square miles and contains a number of storage basins with an aggregate capacity of 1,200,000,000 cubic feet, averaging about 7 miles distant from the Quaker Bridge Lake and 300 feet above its level.

A new dam is now building which will increase this capacity to upwards of 1,800,000,000 cubic feet.

The greatest recorded flood of the river, measured at Croton Dam, is 1,070,000,000 cubic feet in 24 hours.

Most dams of great height are built of stone, laid in hydraulic mortar. This is the class of work recommended by recent writers upon the subject. The three profiles presented to us for consideration are proportioned for masonry of this kind, and we understood that its use for Quaker Bridge Dam had been determined upon by the Aqueduct Commissioners. We have therefore limited our studies to dams so built.

Our first discussions related chiefly to the forces, whether usual or exceptional, that might be brought to bear upon the structure. These were classed under four general heads:

(1) The quiescent and ever-acting forces, such as the weight of the masonry and the pressure produced by the impounded water.

(2) Forces produced by the expansion of ice in place, or by floating masses.

(3) Forces produced by waves of translation, the possible cause of such waves being the giving way of a dam above or an extensive land-slide.

(4) Earthquake shocks.

Quiescent Forces.—It was determined that the specific gravity of the masonry should be taken at 2.34, making the weight of a cubic foot equal to 2.34 times 62.5 pounds, or 146.25 pounds.

Krantz assumes a specific gravity of 2.3, or a weight of 143.75 pounds per cubic foot for masonry built of hard stone (granite or limestone).

The experiments of M. Bouvier upon granite rubble led him to adopt a weight of 147.3 pounds per cubic foot.

While building Boyd's Corner Dam on the Croton River, a careful account was kept of all the materials entering into its construction, from which account the specific gravities of the various classes of masonry were computed. These varied from 2.13 to 2.71, and the specific gravity of the whole mass was found to be 2.34, and we have thought it best to adopt the same specific gravity for the Quaker Bridge Dam.

The aggregate length of the spillways will be about 1300 feet. A depth of about 2.25 feet on the crest would pass the largest recorded flood in the valley, and it will be only on rare occasions that the water can reach the elevation of 202 feet above the tide.

This elevation for the water surface, as producing what may be termed the maxi-

mum quiescent stresses, has been adopted in computing the pressures which the masonry throughout the body of the dam must resist.

The wasteway, or channel for carrying off the surplus waters from the surface of the reservoir, will be constructed in rock cuts and over subsidiary dams so situated that the overflowing water will not touch the main dam.

Ice.—In our search for information upon the expansive force of ice in place, caused by increase of temperature, we found little of value recorded; but we obtained valuable, though somewhat conflicting, information by correspondence and personal interviews, which information, supplemented by experimental data, concerning its strength, elasticity, and rate of expansion under a rising thermometer, has led us to the opinion that the dam should be proportioned to resist a thrust at the highest ice line of about 43,000 pounds per lineal foot.

More positive information was available regarding the force exerted by ice-floes. Under certain unfavorable conditions, where ice-jams form in a quick-running current, it appears to be almost irresistible by direct opposition. But as, in the case of the Quaker Bridge Dam, the water current, when there is one, will tend to divert the floes away from it, and direct impact can be produced only by sheets of ice driven by the wind, we have concluded that, if the dam be proportioned to resist the pressure of 43,000 pounds per lineal foot, above mentioned, it will be of ample strength to withstand the attack of floating masses.

Waves of Translation.—To secure the dam from injury by waves of translation, its upper portion, where the effect of such waves would be greatest, has been so designed as to give a coefficient of at least 2 against overturning, when the water level may be at an elevation of 214 feet above tide, or at the top of the parapet.

Earthquakes.—Earthquake shocks may vary from a slight tremor to an immeasurable force. The dam, if proportioned to resist the forces before considered, will have ample stability to withstand all but shocks of the severest nature. Probably of all the considerable structures in the region affected by such an earthquake it would be the last to succumb.

The Profile or Cross-section of the Dam.—To resist these forces, or at least those of them which may be considered measurable, we have agreed:

(*a*) That the coefficient against overturning should, at all points, be not less than 2;

(*b*) That the ratio of the weight of the masonry above any horizontal plane or joint, to the maximum force tending to cause sliding or shearing along the plane, should not be less than 3 to 2;

(*c*) That the maximum quiescent stress on the down-stream end of the joints at the elevation of the river-bed, 35 feet above tide, should not exceed 10 tons per square foot (139 pounds per square inch);

(*d*) That below that elevation, where the strength of the masonry to resist crushing is aided by the lateral pressure of the earth, the maximum quiescent stress should not exceed 14 tons per square foot (194.5 pounds per square inch); and

(*e*) That the pressures upon the joints of the up-stream face may be somewhat greater, since they will be permanently reduced as soon as the reservoir begins to fill.

We agree in judging it prudent that in so important a structure as the Quaker Bridge

Dam these conditions should be fulfilled, and we believe that, if fulfilled, the cross-section will be amply strong for the functions it will be called upon to perform.

The profile designed by the Engineers of the Aqueduct Commissioners, and submitted to us by the Commissioners, does not meet the requirements which we think should be met for complete safety. We were therefore, under our instructions as we understood them, called upon to prepare a profile which we could recommend for adoption. We have prepared such a profile, and herewith present it under the title Profile N. (See Plate LXXIV.)

Comparing this profile with that of the Aqueduct Engineers, which we have designated Profile Y* (see Plate LXXIII.), the chief point of difference is in the greater thickness of N in the upper portion of the dam. This increase of thickness appears necessary to resist the shock of ice and excessive freshets. The amount of masonry above the plane 100 feet below the level of the flow line of the reservoir will be about 40,000 cubic yards greater by Profile N than by Profile Y. . . .

We have given the plans laid before us, and the arguments presented to us relative thereto, attentive consideration, covering a field of study so extensive that it has seemed advisable to present herein only the conclusions upon which we are agreed, and not to spread before the Commissioners the method by which they have been reached, or a discussion of the several arguments in detail.

As to curved and straight plans generally, without reference to the Quaker Bridge location, all authorities agree that the same principles should be followed in the designing of the profile, whatever the plan, unless the curve has a very short radius, not exceeding, say, 300 feet.

In studying the transmission of pressures through the masonry of a dam built on a curved plan and subjected to water pressure on one side, we have made calculations of their magnitude, which, while only roughly approximate and showing limits which probably are not exceeded, rather than actual values, yet have appeared to us of sufficient weight to materially aid in reaching just conclusions.

Our conclusions may be thus stated:

(1) That, in designing a dam to close a deep, narrow gorge, it is safe to give a curved form in plan and to rely upon arch action for its stability; if the radius is short, the cross section of the dam may be reduced below what is termed the gravity section, meaning thereby a cross-section or profile of such proportions that it is able, by the force of gravity alone, to resist the forces tending to overturn it or to slide it on its base at any point.

(2) That a gravity dam, built, in plan, on a curve of long radius, derives no appreciable aid from arch action so long as the masonry remains intact; but that, in case of a yielding of the masonry, the curved form might prove of advantage.

The division between what may be called a long radius and what may be called a short radius is of course indefinite, and depends somewhat upon the height of the dam. In a general way, we would speak of a radius under 300 feet as a short one, and one of over 600 feet as a long one, for a dam of the height herein contemplated.

(3) That, in a structure of the magnitude and importance of the Quaker Bridge Dam, the question of producing a pleasing architectural effect is second only to that of structural stability, and that such an effect can be better obtained by a plan curved regularly on a long radius than by a plan composed of straight lines with sharp angular deflections.

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* This profile is shown by the dotted lines in Plate LXXIV.

(4) That the curved form better accommodates itself to changes of volume due to changes of temperature.

While danger of the rupture of the masonry of the dam by extraordinary forces, if built on the profile herein recommended, is, in our opinion, very remote, yet it exists; and because it exists, and because the curved form is more pleasing to the eye, better satisfies the mind as to the stability of the structure, and more readily accommodates itself to changes of temperature, we think that it should be preferred in any case where it would cause no great addition to the cost.

In comparing different locations of the dam, in order to discover the one which combined most effectively the advantages of economical construction and pleasing effect, we were confronted with the fact that our calculations indicate that, in a dam built upon a curved plan of large radius, the bottom down-stream toe pressures are increased beyond those in a straight dam of the same section, in consequence of the length of the toe being less than the length of the face to which the pressure of the water is applied.

This increase of pressure is not exactly proportional to the decrease of length of toe, but is of such magnitude that it should not be neglected in designing the section of the dam; and it involves the necessity of increasing the mass of masonry in a certain proportion to the radius of the curvature. . . .

Conclusions.—In view of the premises and pursuant to our instructions, and believing that the dam will be more pleasing in appearance and better able to resist extraordinary forces if built on a curved plan, and bearing in mind that an excessive thrust in the direction of the curve cannot be produced until the force of gravity has been overcome, and that the profile N is so proportioned that more than twice the greatest pressure exerted by any conceivable ordinary force is necessary to overcome the resistance of gravity, we recommend the adoption of the Profile or Cross-section N, and of a curved plan on a radius of about 1200 feet as hereinbefore described, and we advise that the exact line be determined after further borings shall have established the most desirable location on the conditions prescribed.

It should be added, in conclusion, that the form and dimensions herein recommended for adoption are prescribed on the assumption that the structure shall be well founded, and that its material and workmanship shall be of the first class in their several kinds.

Respectfully submitted,

JOS. P. DAVIS,
J. J. R. CROES,
WM. F. SHUNK.]

The New Croton Dam (Plates LXXV. to LXXIX.).—In the preceding pages we have given an account of the plans prepared for the "Quaker Bridge Dam." Owing to the opposition made to this project, the Aqueduct Commissioners finally decided to build the dam about 1½ miles further up-stream, at a place known from a house near by as the "Cornell site." This dam, which has been named the New Croton Dam, is now being constructed.* It will consist of three parts:

1. A central masonry dam about 730 feet long.

2. A masonry overflow-weir, about 1000 feet long, on the north side of the masonry dam and nearly at right angles thereto.

* The specifications for the work are given in the Appendix on page 199.

3. An earthen dam with a masonry core-wall, about 440 feet long, forming a continuation of the masonry dam to the south side of the valley.

At the junction of the earthen and masonry dams a large masonry wing-wall will be built. The overflow-weir, masonry dam, and core-wall of the earth bank are all founded on rock and will form a continuous masonry wall across the valley, which has a width of about 450 feet at a level about 25 feet above the old river-bed, and of about 1300 feet at the elevation of the top of the earth bank.

It was expected that less height would be required for the dam at the Cornell site than at Quaker Bridge. Owing, however, to deep pockets and the depth to which the bed-rock had to be removed in some places before a satisfactory foundation was obtained, the New Croton Dam will have a maximum height of about 290 feet.

The profile adopted for the dam is practically the one designed by the engineers of the Aqueduct Commission for the proposed Quaker Bridge Dam (Plate LXXIII.), the only difference being that the polygonal outlines have been rounded off by curves.

The overflow-weir is being built according to the stepped profile shown in Plate LXXVIII., the steps varying from 1.5 to 9 feet in rise and from 2 to 6 feet in tread. It is located about parallel with the contour-lines on the north side of the valley, and is curved to join the main masonry dam. Its height will vary from 10 feet at the end on the hill-side to 150 feet at the junction with the main dam. Its crest will be 4 feet below the high-water mark of the reservoir. A waste channel—50 feet wide at the beginning and 125 feet wide at the masonry dam—has been excavated to carry off the overflow from the reservoir, which it will lead to the old river channel below the dam.

The profile designed for the earthen dam is shown in Fig. 1, Plate LXXIX. According to the original plans this dam was to have a maximum height of about 120 feet above the surface. It was decided, however, to extend the central masonry dam about 110 feet further south than originally contemplated. Owing to this change the maximum height of the earthen dam will be only about 80 feet. The trench for the core-wall had to be excavated in some places to a great depth to obtain a rock foundation. At the point where it reaches its greatest height the core-wall was founded 136 feet below the surface and carried 50 feet above it, making a total height of 186 feet. The wall had a top-width of 6 feet. Both faces are battered uniformly to a depth of 136 feet below the top, where the wall is 18 feet thick. From this point to the foundation the faces are vertical.

The top of the earth dam will rise 10 feet above the crest of the masonry dam. Both sides will be sloped 2 to 1, the down-stream slope being broken, however, by two berms, 5 feet wide, made respectively 30 and 60 feet below the top of the dam. The berms will be ditched and paved to carry off rain-water. The up-stream slope of the dam is to be protected by a stone paving 2 feet deep placed upon 18 inches of broken stone. This paving will extend to a level 10 feet above the high-water mark. The top of the dam (except where a roadway is to be formed), the up-stream slope above the paving, and the down-stream slope will be covered with good soil and sodded.

Gate-houses.—The flow from the New Croton Reservoir will be controlled by a large gate-house which has been constructed at the inlet into the New Croton Aqueduct,

about 3 miles up-stream from the site of the New Croton Dam. The Old Croton Aqueduct, which was built along the south side of the valley, will be submerged when the New Croton Reservoir is filled. The Old Aqueduct has been connected with the large gate-house just mentioned, and will be controlled by a small gate-house which is being built at the south end of the earthen part of the New Croton Dam, adjoining the core-wall. The Old Aqueduct crosses the dam at this point. By means of the small gate-house the flow in the Old Aqueduct will be regulated, which can be made to convey water directly to New York or to serve merely as a conduit leading water from a point near the dam to the large gate-house at the inlet of the New Croton Aqueduct.

The substructure of the small gate-house at the dam will be divided into four water-chambers, the two easterly ones serving as inlet-chambers. One of these will be connected with the Old Croton Aqueduct; the other will draw water near the new dam by means of three short oval conduits (6 feet wide by 10 feet high), forming a bottom, middle, and top inlet. These openings can be closed at the gate-house by means of stop-planks placed in grooves in the side-walls.

The two westerly chambers of the gate-house will be connected by openings controlled by sluice-gates. The outlet will take place from the southwest chamber, to which the Old Aqueduct will be connected. A system of 12-inch pipes, placed below the floor of the substructure, will serve for draining the water-chambers.

A larger gate-house will be built on the up-stream face of the masonry dam, where it joins the overflow-weir. This gate-house will merely serve for drawing down the reservoir by discharging the water into the old river channel below the dam. Its substructure will be divided by brick walls into three separate water-chambers. Each chamber will have an inlet opening about 30 feet above the original river-bed and just above the embankment that will be made above the dam with the surplus material excavated from the foundation-trench. Sluice-gates for regulating the flow from the reservoir and stop-planks for closing the inlets will be provided for each water-chamber. Three 48-inch cast-iron outlet-pipes (one for each water-chamber) will be laid in the masonry of the dam. These pipes will be controlled by stop-cocks, which will be placed in a vault constructed on the down-stream face of the dam. The outlet-pipes will discharge the water into the old river-bed a short distance below the dam.

Details of Construction.—The hearting of the overflow-weir and masonry dam will consist of rubble masonry, faced with ashlar. The courses of the facing-stones (which begin for the overflow at the foundation, but for the main dam only above the refilling which is placed in front and back of the dam) will vary in rise from 30 to 15 inches. The joints for this work are not to exceed $\frac{1}{2}$ inch for 4 inches from the exposed face, and are not to be over 2 inches wide for the remaining depth. The ashlar is to have a minimum depth of 28 inches. In each course every third stone is to be a header, having a length of at least 4 feet. The stretchers will vary from 3 to 7 feet in length. The headers and stretchers are to alternate approximately in the successive courses.

The steps on the down-stream side of the overflow are being made of block masonry, having generally a greater rise and width than the facing-stone, and a sufficient depth

to bond under the next step above. The joints in this work are not over one inch wide. For the upper step over which the water passes first, the coping is to be made of granite dimension-stone, having the exposed faces roughly pointed.

The heavy cornice of the masonry dam will be made of granite dimension-stone. The roadway on top of this structure will be formed of concrete covered with asphalt. It will be drained by short pipes, placed under the coping. The roadway will be continued over the earth dam. A highway bridge, of about 230 feet span, will carry the road over the waste-channel to the north side of the valley. The large wing-wall at the southern end of the masonry dam will be built of rubble, which will be faced above ground with ashlar and coped.

The water-chambers of the gate-houses will be lined with brick laid in Portland cement mortar, except at the gate-openings, where granite dimension-stone will be used. The grooves for the stop-planks will be formed by castings joined into the brick lining.

Protection Works.—Owing to the great depth to which the foundation-trench for the masonry dam had to be excavated, expensive works were required for turning the river from its former course. A new channel for the river, 125 feet wide and about 1100 feet long, has been excavated in the rock on the north side of the valley. To avoid expense it was kept 5 feet higher than the old bed. The river is confined in its new channel on the north by the slope of the hill, and on the south by a masonry wall continued at both ends by earthen dams which extend across the old channel, the upper one serving to turn the river into its new course.

The wall is built for about 300 feet on each side of the centre-line of the dam; it is 3 feet wide at the top and 13 feet at the base, its height being 23 to 25 feet above the grade of the new channel. The face towards the water channel is almost vertical. On the other side of the wall (except where it crosses the site of the dam) an earth embankment has been carried up for about half its height. Some portions of the wall which form permanent work have been made stronger than the dimensions given.

The masonry wall is being continued at each end by an earthen dam, 10 feet wide on top and about 30 feet high. Towards the channel the banks are sloped $1\frac{1}{2}$ to 1, and on the opposite side 2 to 1. Water-tightness is insured in these earthen dams by providing them with a core-wall formed of two courses of 3-inch tongued-and-grooved sheet-piles, which extend 3 feet below the top of the banks to about 20 feet below the original surface. The two courses of sheet-piles are spiked together and are stiffened above the river-bed by frequent courses of horizontal range-timbers, which were fastened to the sheeting as it was put in place. The toe of the slopes on the channel side is formed of heavy cribwork 10 to 12 feet high. In both dams two cribs, each 10 feet wide, are placed 6 feet apart, the space between them being filled with compact earth. The cribs are joined together by frequent cross-ties, extending through the 6-foot spaces. The outer faces of the cribs and those on each side of the filling just mentioned are covered with 3-inch tongued-and-grooved sheeting, sunk $3\frac{1}{2}$ to 10 feet into the ground below the bottom of the crib. The cribs are to protect the toe of the embankments against the scouring action of

the water, which may have a depth of 15 to 19 feet during great freshets. The total length of the masonry and earth dams which bound the new channel of the river on the sound is about 1600 feet.

The Construction.—The contract for the construction of the New Croton Dam was let to James S. Coleman on August 31, 1892. Some other contractors acquired an interest in the contract at various times. The work is now being conducted by Coleman, Breuchaud & Brown, the senior member of the firm being the original contractor.

The work accomplished to January 1, 1899, has been as follows: The river has been diverted into its new channel and the protection works have been built. The trench for the masonry dam has been excavated and the main masonry dam has been carried up to a level of about 4 feet below the old river-bed. A large amount of masonry has been laid in the overflow-weir and core-wall. Some conception of what has thus far been accomplished may be formed from the following items of work done to January 1, 1899:

Earth excavation, ...	1,059,000	cubic	yards.
Rock excavation, ...	296,000	"	"
Masonry of all kinds, ...	340,000	"	"

The total amount of work done to the above date was estimated at $2,876,000.

All the stone laid in the dam except the granite required for trimming, coping, etc., is obtained from a quarry situated about 1½ miles from the work. The stone is transported on a narrow-gauge railway in cars hauled by locomotives. The stones are laid in the wall either by derricks or by means of three cables which have been stretched across the valley above the wall. The biggest cable is 2½ inches in diameter, the distance between its supports being about 1200 feet. American Portland cement mortar, mixed 2 to 1, was used for the course of masonry next to the bed-rock. For all the other courses of masonry ordinary American cement mortar mixed 2 to 1 was used except in winter (November 1 to April 1), when the masonry was all laid in Portland cement mortar mixed 3 to 1. The cement is delivered in bags. Three one-story wooden cement-sheds have been erected near the dam. They are capable of storing about 14,000 barrels of cement. About 16,000 barrels of cement were used in laying the maximum amount of masonry in one month, which amounted to 17,000 cubic yards. Some of the masonry has been laid during freezing weather, the sand being heated by being piled over steam-pipes and all frost being removed from the stones by a steam-jet. Salt was added to the water used for mixing the mortar. No masonry was laid when the temperature was below 20° Fahrenheit.

The greatest force employed on the work at any time was about 800 men and 30 teams.

Contractor's Plant.—Large pumps were required to remove the water which flowed into the main foundation-trench from springs in the rock, leakage from the new river channel, and drainage from the surface. The main pumping-plant consists of three Worthington compound pumps, each having a nominal capacity of 4,000,000 gallons in 24 hours with 80 pounds steam and against a head of 90 feet. Only two of these pumps have been used at a time, the third being always kept in reserve. The pipes

through which these pumps discharge the water directly into the river are 12 inches in diameter. Smaller pumps were used for raising the water from the deepest sumps to those of the main pumps. Two 8-inch Bush centrifugal pumps, one 10-inch and two 6-inch Worthington pumps, besides small steam-siphons, were used for this purpose. The amount of pumping required has not exceeded 5,000,000 gallons a day for any length of time, but it has been more for short periods. Steam is supplied to the pumps by four 100 horse-power boilers, which are located on the bank of the river beyond the slopes of the foundation-trench.

Besides these boilers there are 12 others, aggregating about 530 horse-power, which furnish the steam required for operating the cable-ways, derricks, etc. The narrow-gauge railroad is equipped with 7 locomotives and 83 flat cars, each 20 feet long and having a capacity of 20 tons. Thirty derricks, each having a Lidgerwood double-drum hoisting-engine, are in use on the dam. Thirteen similar derricks, with separate hoisting-engines are used in the quarry. All the engines, drills, etc., used in the quarry are supplied with steam from 6 boilers having together a capacity of 280 horse-power.

To keep the large plant described above in order, the contractors had to erect a machine-shop in which an electric plant has also been placed.

The contractors have leased a private dock at Croton Landing on the Hudson, where they unload most of their supplies, which are hauled by teams to the site of the dam, a distance of about 2½ miles. A large amount of coal is always stored on the dock before winter begins.

Engineers.—The plans for the New Croton Dam were all prepared by Mr. A. Fteley, the Chief Engineer of the Aqueduct Commission. Mr. Charles S. Gowen has been in charge of the work from the beginning as Division Engineer. Mr. B. R. Value has been the Assistant Engineer in immediate charge of the work on the dam.

PART II.

CHAPTER I.

EARTHEN DAMS.

AT a very early period of history the construction of earthen dams to impound water was begun. Many reservoirs constructed in this manner in India, ages ago, are still in use. They are called "tanks." One of them—the Veranum Tank—which covers 35 square miles, is formed by an earthen dam 12 miles long. The Poniary Tank, which is no longer in use, had a water area of 60 to 80 square miles. Its dam had a length of about 30 miles.*

These old dams, which have withstood so successfully the ravages of time, were constructed in a very primitive manner. They are simply large mounds of argillaceous earth which was brought in baskets to the site of the dam and was compacted by the tread of the army of workmen engaged on the work. Their profiles are much larger than those adopted for modern dams.

By the experience of centuries and the lessons taught by many catastrophes the proper dimensions of earthen dams and the precautions that should be observed in their construction have been fully established. The design of such works should not be based upon mathematical calculations of equilibrium and safe pressure, as in the case of masonry dams, but upon results found by experience. Most of the earth dams constructed within the last century have had a large margin of safety in resisting the water-pressure, both as regards overturning and sliding, and yet frightful distasters, such as the rupture of the Dale Dyke and the Johnstown dams (see page 123), have resulted from faults in designing some details or from neglect in the construction of the work.

General Plans.—An earthen dam may consist of—

1. A homogeneous bank of earth.
2. A bank of earth having a puddle-core (Plate LXXX.).
3. A bank of earth having a masonry core-wall (Plate LXXIX.).
4. A bank of earth having puddle placed on the water slope.

The first method of construction can only be safely used when a sufficient quantity of earth or gravel containing enough clay to make the dam water-tight can be obtained at a reasonable cost.

* See "The Designing and Construction of Storage Reservoirs," by Arthur Jacob, B.A.

Where it would prove too expensive to form the whole dam of such binding, water-tight material, only a central core is formed of clayey earth or gravel, ordinary earth being used for the other portions of the dam. In this case the whole water-tightness of the dam depends on the core, which is formed of "puddle material," viz., clay, earth, and sometimes gravel, which are thoroughly mixed and compacted. Even when good material is available for the dam, puddle should be placed below the surface in a trench excavated to an impervious stratum, in order to prevent leakage under the dam.

The third plan consists in substituting a masonry "core-wall" for the "puddle-core." This plan will have to be adopted when no clayey earth can be obtained at a reasonable cost. It will generally be found more expensive than plans 1 or 2, but, on the other hand, has great advantages as regards safety. For small or temporary dams the core is occasionally made of planks driven as sheet-piling.

The puddle is sometimes placed on the inner slope with a view of preventing the water from percolating into the dam, but this plan is open to two objections: 1st. The puddle is apt to be injured by the settling of the slope, which is sure to occur to a greater or less extent; 2d. Cracks will appear in the puddle when it is exposed to alternate wetting and drying due to fluctuation in the level of the water.

It is important to prevent the water from percolating to the centre of the dam, but any water that may reach there should be drained off so as not to saturate the outer slope. This may be accomplished, when there is a difference in the quality of the earth put in a dam, by placing the most water-tight material in the inner slope, while the more pervious kinds (gravel, etc.) are deposited in the outer slope. Drain-pipes are occasionally laid at the outer side of the puddle-core to carry off any water that may percolate through the puddle.

In some dams selected water-tight earth is placed on both sides of the puddle-core, the remaining parts of the embankment being made of more pervious material.

Materials.—The best material for forming an earthen dam is an earth, a gravel, or a hard-pan containing just enough clay to give it the required water-tightness and binding quality. Clay alone or in a large proportion will not answer the required purposes, as it swells when wetted and shrinks in drying. This quality makes it very dangerous in any part of a dam where it may be alternately wet and dry. Gravel will tend to fill up a hole that may be formed in a dam, but clay is apt to arch over an opening, which may be enlarged and lead to the rupture of the structure. Many failures have been caused by using too much clay in the body of the dam. Some authorities recommend the use of 20 to 30 per cent of clay for the body of the dam or in the puddle-wall, but a much smaller quantity—5 to 20 per cent—will often be found to be sufficient. No general rule can be laid down, as the percentage of clay depends upon the nature of the materials with which it is mixed.

The Profiles of earthen dams are determined entirely by practical considerations. The general dimensions of the profile depend upon the materials used and the height of the bank, additional strength being given to high dams. The dimensions usually adopted are as follows:

PROFILES FOR EARTHEN DAMS.

Top-width,	10 to 30 feet.
Superelevation above high water,	5 to 25 feet.
Inner (up-stream) slope,	2 : 1 to 3 : 1.
Outer (down-stream) slope,	$1\frac{1}{2}$: 1 to $2\frac{1}{2}$: 1.

The top-width should be at least 10 feet. If the top of the dam is to serve as a road across the valley, it may require a width of 20 to 30 feet, but the latter figure need rarely be exceeded. The top of the dam should be sufficiently raised above the highest water-level to be beyond the reach of the highest waves in the reservoir. The height of the waves depends upon the extent and depth of the reservoir and upon its exposure to winds. Where violent winds occur waves may be dashed against a dam to a height of 10 to 15 feet.

Ordinary earth will stand on a natural slope of $1\frac{1}{2}$ to 1. The outer slope of the dam may be given this inclination, but it is usually made a little flatter, viz., 2 to 1 and even $2\frac{1}{2}$ to 1. In dams of considerable height (60 to 100 feet) the outer slope is often broken by one or more berms, placed about 30 feet apart vertically, in order to increase the cross-section and base of the dam and to prevent any washing of the long outer slope by heavy rain-storms. The berms should be provided with paved gutters which lead the rain-water to the hillsides of the valley (Fig. 1, Plate LXXIX.).

Earth saturated with water assumes a much flatter slope than when dry. For this reason the inner slope of the dam is made flatter than the outer one, viz., $2\frac{1}{2}$ to 1 or 3 to 1. The latter slope is usually adopted by English engineers.

The inner slope is protected against the action of the water and against vermin by a paving of rectangular stones having a thickness of 15 to 24 inches, according to the height and importance of the dam. This paving should be placed on a 12- to 18-inch layer of broken stones (about 2 to 3 inches in diameter), and should be carried up 5 to 15 feet above the high-water level, as the height of the waves may require.

The top of the dam, the outer slope, and the inner slope above the paving are covered with good soil and sodded.

The Puddle-core, required when Plan No. 2 is adopted, should be made of the best materials that can be obtained at a reasonable expense. Gravel, sand, clay, and occasionally peat have been used for this purpose. A clayey gravel or hard-pan is the best material that can be used. Sometimes additional clay will have to be added to the gravel or hard-pan, but the percentage of clay in the whole mass should not exceed the figures mentioned on page 112. Clay should never be used alone for the puddle-core, as it gets slimy and sticky when wet and cannot be spread uniformly. In drying it shrinks and cracks and may still retain water. The clay should be free of sand and soft stones. The materials used in the puddle-core should be uniformly mixed, sufficiently moistened and worked ("tempered") to make a tough, elastic mass which should be carefully deposited in layers. If the work is interrupted the puddle should be covered with boards or earth to prevent it from cracking by drying too rapidly.

The thickness of the puddle-core depends upon the kind of material of which it is composed and upon the "head" of the water to be resisted. Considerable variation occurs in cross-sections adopted by different engineers for these walls. The puddle-core should be 4 to 8 feet thick at the highest water-level. Both faces should be battered uniformly so that the thickness of the wall at the natural surface shall be one third of the head of the water to be resisted. From the natural surface to the bottom of the foundation-trench the thickness of the puddle-core is either made uniform or gradually diminished, but never over 50 per cent; i.e., the thickness at the bottom of the trench should be at least one half the thickness at the natural surface. This reduction of the thickness of that portion of the puddle-wall which lies below the surface of the ground is really contrary to theoretical requirements. It is due to the practical difficulties encountered in trying to excavate a deep trench with vertical sides. Offsets in the sheeting are almost sure to occur. This would ordinarily necessitate starting the foundation-trench with a great width, involving much expense, if the puddle-core were required to have the same thickness from top to bottom of the foundation-trench.

The puddle-core must be founded on an inpervious stratum (rock, hard-pan, clay, etc.) which will make it impossible for the water to percolate under it. To reach such a stratum the foundation-trench will often have to be excavated to a great depth. A covering of at least 3 to 4 feet of ordinary earth must be placed on top of the puddle-core to protect it against frost. This object is accomplished on the sides by the slopes.

Mr. J. T. Fanning,* M. Am. Soc. C. E., has used for some dams a very superior kind of puddle composed of coarse gravel, fine gravel, sand, and clay. The voids of the coarse gravel are filled with fine gravel, the voids of the resulting mixture are filled with sand, and enough clay is added to give the mixture sufficient binding quality. This puddle is nearly free of voids. It weighs almost as much as granite and resists not only the action of water, but also the attacks of rats, eels, and other vermin.

The following table gives the theoretical proportions required for this puddle and those used by Mr. Fanning in practice:

PUDDLE OF GRAVEL, SAND, AND CLAY.

Materials.	Percentage of Voids.	Cubic Yards Required.	
		Theoretically.	Practically.
Screened coarse gravel..............	28–30	1.00	1.00
Fine gravel	30	0.28	0.35
Sand............	33	0.08	0.15
Clay...	....	0.03	0.20
Total of materials....................	....	1.39	1.70
Resulting puddle.	....	1.00	1.30

* Treatise on Water-supply Engineering, by J. T. Fanning. New York, 1882.

Fanning describes the manner in which he formed such a puddle-core for a dam as follows:

"When measured by cart-loads, the quantities became eight loads* of mixed gravels, one load of sand, and two loads of clay, the cubic measure of each load of clay being slightly less than that of the dry materials. The gravel was spread in layers of 2 inches thickness, loose, the clay evenly spread upon the gravel and lumps broken, and the sand spread upon the clay. When the triple layer was spread, a harrow was passed over it until it was thoroughly mixed, and then it was thoroughly rolled with a 2-ton grooved roller, made up in sections, the layer having been first moistened to just that consistency that would cause it to knead like dough under the roller, and become a compact solid mass.

"The proportions adopted for the core were a thickness of 5 feet at the top at a level 3 feet above high-water mark, and approximate slopes of 1 to 1 on each side."

The puddle described by Mr. Fanning is certainly excellent, but, considering the amount of work and care required in mixing it, we doubt whether it is in ordinary cases much cheaper than concrete or rubble masonry.

Masonry Core-walls are doubtless the best means of insuring water-tightness in an earthen dam, and should generally be adopted when the means at disposal will permit their use. They can be constructed of concrete or of rubble masonry. The up-stream face should be well plastered with cement mortar.

These walls are not designed to resist the whole water-pressure in the reservoir, as they are simply to act as cut-off walls that will stop any water which may have percolated through the inner slope. Theoretically we would conclude that if any water reaches the core-wall, a section of it might eventually have to resist the whole hydrostatic pressure from the reservoir. As the wall is only backed by earth, we would imagine that the light core-walls adopted in practice might fail, but experience does not prove this to be the case. Tests made by sinking holes along the inner face of core-walls will generally show that water, having a few feet less head than that due to the reservoir, reaches the wall. Nevertheless these walls stand, and we must, therefore, conclude either that they are never subjected to any extent to the full pressure due to the reservoir, or else that the well-rolled earth with which they are backed enables them to resist the pressure.

Core-walls for high dams are usually given a stronger section than those for lower ones. The top of the wall, which should be placed at high-water level, is made $2\frac{1}{2}$ to 6 feet wide. Both faces are battered uniformly from the top to the surface of the ground and are then vertical to the foundation, which must be laid on an impervious stratum. The thickness of the core-wall at the natural surface should be about $\frac{1}{6}$ to $\frac{1}{7}$ of the "head" in the reservoir. Instead of battering the faces the increase in thickness may be made by offsets, about 10 feet apart.

A Waste-weir (Overflow-weir, Spillway) to discharge the water which rises above the high-water level must be provided for every reservoir. Its crest is placed 5 to 25 feet below the top of the dam, according to the superelevation given the latter.

* Seven loads of coarse and three loads of fine gravel make, when mixed, about eight loads in bulk.

The waste-weir usually forms part of the main dam, but occasionally the flood-water may be discharged into a lateral valley by excavating a low ridge or by building the waste-weir as an auxiliary dam at some depression lying below the high-water mark. When the material excavated to form a waste-weir consists of rock, it is levelled at the proper height with concrete or rubble. If the excavation is in earth, a suitable overflow-wall must be built.

In the case of a reservoir supplied by a small watershed the waste water may be carried off by means of a well located in the reservoir, usually constructed in the gate-house controlling the outlet.

When the waste-weir forms part of the main dam it is placed near the centre of the valley, if both side hills are of earth. If either of them consists of rock at or near the surface, the waste-weir is formed at the rocky side of the valley by excavating the rock or building a low wall on it, as its elevation may require.

The determination of the proper length of a waste-weir is a very important matter. Many a dam has failed because its waste-weir could not discharge an unusual flood. The amount of water which a weir of a certain length will discharge with a given depth of water can be calculated by the following formulæ given by Mr. J. B. Francis for depths of 9 to 36 inches:

For wide-crested weirs

$$Q = 3.012lH^{1.53}.$$

For flashboards with square edges

$$Q = 3.33(l - 0.1nH)H^{\frac{3}{2}};$$

in which $Q =$ discharge in cubic feet per second;

$l =$ length of the weir in feet;

$H =$ depth of water above the crest, in feet, measured above the weir;

$n =$ number of end-contractions.

No general rule can be given for estimating the maximum amount of flood-water that may reach a reservoir from a given watershed. This quantity will depend upon the rainfall, and the character and extent of the watershed. The larger the watershed, the longer will be the period required for the water flowing off, after a certain rainfall, to reach the reservoir.

An empirical English rule for watersheds of less than 3 square miles area is to allow 1 lineal yard of waste-weir for every 100 acres in the watershed. For watersheds having areas of 1 to 50 square miles E. Sherman Gould, M. Am. Soc. C. E., recommends in his book on "High Masonry Dams" an allowance of 1 lineal yard of overflow-weir per square mile of watershed. He suggests, also, the following empirical formula:

$$\text{Length of overflow in feet} = 20\sqrt{\text{number of square miles in watershed}}.$$

Whenever it is possible, the engineer should collect data as regards the maximum amount of flood-water likely to reach a reservoir. The greatest recorded depth of

water in the river at some bridge or culvert above the reservoir may furnish valuable information in this connection.

In calculating the length of the waste-weir, the depth of the sheet of water passing over the weir in the worst recorded flood is usually assumed as 1 to 3 feet. As the top of the dam is generally 5 to 15 feet above the crest of the overflow-weir, the flood-water may rise considerably above the assumed highest level without endangering the safety of the dam.

To increase the capacity of a reservoir, the height of a waste-weir is sometimes temporarily raised by means of planks (known as "stop-planks" or "flashboards") placed in grooves in iron standards, or masonry piers built on top of the weir, or by forming a small earthen bank on top of the weir. In the latter case no danger could arise from an unexpected freshet, as the temporary earthen bank would be washed from the top of the overflow-weir. In the former case, the stop-planks must be removed either by hand or by some automatic contrivance, during floods. There is, however, always the danger of the planks not being removed in time.

A suitable channel must be constructed for conveying the waste water of the reservoir from the overflow-weir to some point below the dam where it can be turned into the stream. It is important that this channel should have sufficient capacity to discharge easily the maximum amount of water that may flow over the waste-weir.

A By-wash is often provided for reservoirs supplied by streams liable to carry much suspended matter at times. Its object is to lead the stream during floods around the reservoir, discharging the muddy water either into the stream below the reservoir or into compensating reservoirs in which the quality of the water is of no importance. These basins serve during droughts to feed the stream below the reservoir.

The discharge of the stream into the service or the compensating reservoir may be regulated by sluice-gates. Automatic means are sometimes provided for this object. An ingenious arrangement for accomplishing this purpose, known as separating-weirs (Fig. 17), was first introduced in the water-works of Manchester, England. During

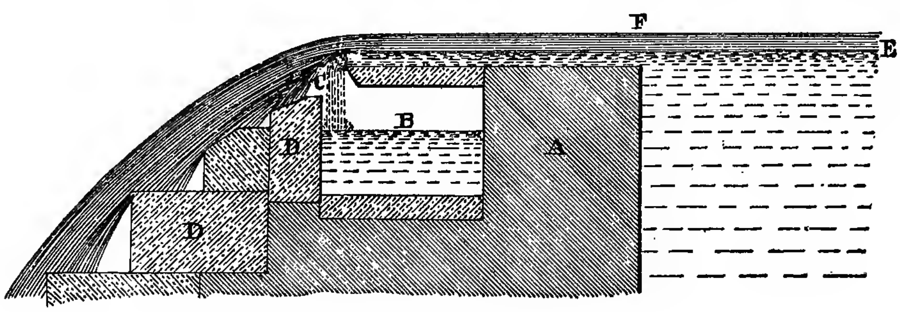

FIG. 17.—SEPARATING WEIR.

floods the water from the gathering-ground is carried, owing to its greater velocity, over an opening of a well into which it falls when its velocity is reduced. The water passing over the opening is led to the by-wash, while that falling into the opening flows to the reservoir. Mechanical means are also sometimes employed for separating the flood from the clear water.

Outlet- and Waste-pipes.—A reservoir is usually provided with two outlet-pipes for conveying the water from the reservoir either directly to the place of consumption or to some stream flowing into a lower storage basin. These pipes are arranged so that one line can be in service while the other is undergoing repairs. A scouring- or waste-pipe is also required for emptying the reservoir for repairs and for "blowing out" deposits of silt, etc. The latter object is accomplished by simply opening the pipe, when the rush and pressure of the water will force out ("blow out") the deposit. The waste-pipe should be located at the lowest part of the reservoir in order to be able to drain it completely. Sluice-gates and stop-cocks for controlling the outlet- and scour-pipes are placed in the gate-house and stop-cock vault described on page 120.

One of the most important details connected with the construction of a reservoir is the manner in which the pipes mentioned above are laid through or around the dam. Formerly it was quite customary to lay these pipes right in the dam, without any protection, the earth being simply packed tightly around them. Numerous failures have proved the danger of this method of construction. The outlet-pipes were frequently cracked by a settling in the dam. The water under pressure which found its way thus into the embankment usually flowed along the smooth outer surfaces of the pipes, washing out a channel which led to the rupture of the dam unless the damage was detected and repaired in time.

The danger of the escaping water flowing along the outside of the pipes can be prevented, to a certain extent, by building at intervals "cut-off walls" around the pipes. They should project at least 2 feet all around the pipes, to which they must be closely fitted. Three or more cut-off walls are usually built.

If flanged pipes are used instead of those of the hub-and-spigot pattern, the flanges will act as cut-offs, impeding the flow of the escaping water. This object may be accomplished in a better though more expensive manner by surrounding the outlet-pipe with a ring of brick masonry.

The only safe method of carrying the outlet-pipes through a dam having a puddle-core is to lay them for their whole length through the dam on a masonry wall built on an unyielding foundation. Masonry piers will not answer the purpose, as a settling may occur between them.

Instead of laying the outlet- and blow-off pipes directly in the dam, they may be placed in a masonry culvert passing through the dam. By this arrangement the pipes are relieved of the pressure due to the weight of the earthen bank, which is borne by the masonry. They can, also, be inspected and repaired. The culvert is usually made circular, but if its section is large it is preferable to make it elliptical, the ratio of the horizontal to the vertical diameter being about as 2 to 3. Sometimes a "horseshoe" section is adopted, as its flat invert gives more space than circular or elliptical sections for placing and repairing the pipes.

As the scouring-pipe should start from the lowest point in the reservoir, the culvert is usually built in excavation. Where it crosses the puddle-trench (which has generally considerable depth) the culvert should be supported by a masonry wall built on an unyielding foundation. As a settling may take place on either side of this

wall it is best to provide the culvert at these points with loose, vertical slip-joints which will permit a settling without the masonry being ruptured.

At its down-stream end the culvert is terminated by arches, strongly buttressed to withstand the pressure of the outer slope and provided with wing-walls. The up-stream end of the culvert is usually joined to the gate-house or water-tower containing the sluices for regulating the flow from the reservoir.

In order to reduce the height of the masonry wall in the puddle-trench, the culvert is usually built descending to this trench and rising from it down-stream. Sometimes, however, the culvert has vertical changes of grade.

Although the outlet-pipes can be safely laid through an earthen dam if the proper precautions are observed, it is best to lay these pipes in a lateral tunnel passing through a hill at either side of the dam whenever this method is not found to be too expensive. The tunnel must be closed at the reservoir by a masonry bulkhead (wall) through which the pipes pass. It should be lined with masonry and left open at its down-stream end so as to permit inspection and repairs of the pipes. During the construction of the dam this tunnel may serve—if large enough—as a temporary channel for the stream.

When an earthen dam is provided with a masonry core-wall, the outlet- and scour-pipes can be safely laid through the dam without being supported by masonry, as the core-wall forms a perfect cut-off which prevents any water from reaching the outer slope of the dam. In this case the water may be safely conveyed through the inner slope in a masonry conduit. Iron pipes laid in a culvert, permitting inspection and repairs, carry the water through the outer slope. The pipes are connected with the masonry conduit by iron reducers * which are built in the core-wall. With this arrangement a settling of the earth either in the inner or outer slope cannot produce any disaster.

Instead of the masonry conduit mentioned above, iron pipes may also be used to convey the water to and through the core-wall. In order to facilitate repairs it is advisable to lay these pipes in a culvert, as in the outer slope. The culvert should be so connected with the inlet gate-house or water-tower that the pipes can be inspected even when the reservoir is full.

The outlet-pipes of low dams are sometimes laid as siphons over the top of the bank. This is the safest arrangement that can be adopted, but the pipes will have to be protected against frost in a cold climate, and a pump will have to be provided to start the siphon when it is stopped by air accumulating at its highest point.

Gate-house (Water-tower, Valve-tower).—The outlet from a reservoir is usually controlled by sluice-gates, valves, or stop-planks placed in a gate-house or valve-tower. For a dam having a masonry core-wall the gate-house is generally constructed at the up-stream face of this wall. If the dam consists entirely of earth, the gate-house is placed in the reservoir near the toe of the inner slope, access to it being obtained by means of a foot-bridge.

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* Iron pipes whose area is gradually reduced. The largest section, at the masonry conduit, is generally made rectangular. This shape is gradually changed to the circular section of the outlet-pipes. The reducers serve to lessen the loss of head at the inlet.

The gate-house may be a very simple structure or one of importance according to circumstances. Fig. 18 shows the plan of a very simple outlet of masonry which contains simply a double set of grooves in the masonry side walls for stop-planks, and has no sluice-gates. The water is ordinarily controlled by stop-cocks placed in a vault at the outer slope, two sets being provided, one placed behind the other. When it becomes necessary to shut off the water at the gate-house, the stop-planks are dropped in the grooves. By tacking a piece of marlin* on the bottom side of each stop-plank quite a tight bulkhead will be formed. Any leaks that may exist can be stopped by calking, and if necessary clay can be dropped between the two sets of stop-planks, but this will rarely be required.

A more complicated gate-house, which may be used for an earth or masonry dam, is shown in Plates LXVI. and LXIX. It consists of a masonry substructure, containing the water-chambers, sluice-gates, etc., and of a superstructure, constructed usually of

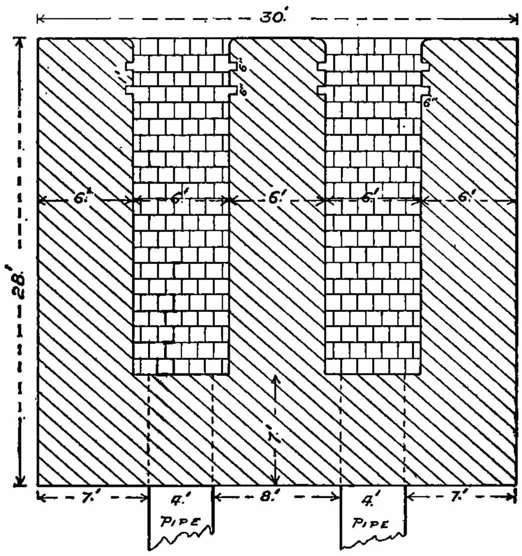


FIG. 18.—OUTLET FROM RESERVOIR.

masonry, which protects the hoisting machinery of the sluice-gates. The substructure is divided by a central wall into two divisions, one for each line of outlet-pipes. A cross-wall divides each division into an inlet- and an outlet-chamber. Each inlet-chamber has three openings at the reservoir for drawing water at different levels, one at the surface † of the reservoir, one at mid-depth, and the third near the bottom. These openings are provided with screens (made of bar iron or fine wire netting, as circumstances may require) and can be closed by stop-planks. One or more openings controlled by sluice-gates are constructed in the cross-walls between each set of chambers. In the back wall of the outlet-chambers the reducers for the outlet-pipes are placed. When a gate-house of this kind is built at the core-wall of an earthen dam, either the inlet-pipes must be laid through the inner slope of the dam to the reservoir, or else the inner slope must be omitted at the gate-house, and wing-walls must be built to retain it on both sides.

* A small tarred cord of two strands used for winding around ropes and cables.

† The best water in a reservoir is usually a few feet below the surface.

Instead of a gate-house of the kind just described, a circular valve-tower is sometimes constructed to regulate the outflow from the reservoir. If the water has but little depth, this tower may consist of a vertical cast-iron pipe, having inlet openings at different levels, and connections, at the bottom, with the outlet-pipes. If the depth of the water in the reservoir is considerable, the valve-tower should be constructed of masonry. The inlet openings may consist of short lengths of pipe embedded in the masonry and controlled by stop-cocks, sluice-gates, flap-valves, or poppet-valves operated from the top of the tower. Plate LXXXI. shows such a tower.

Another arrangement consists in confining the water in a vertical stand-pipe, which is placed in the valve-tower. Short horizontal pipes, placed about 10 feet apart vertically, form the inlet openings and are connected with the stand-pipe. Each inlet-pipe is controlled by a bronze flap-valve, placed just outside the tower, and, also, by a stop-cock or sluice-gate inside of the tower. With this arrangement, the gate-keeper can enter the culvert for the outlet-pipes at the outer slope of the dam and pass into the valve-tower, inspecting the whole outlet-pipe system.

Stop-cock Vault.—In addition to the arrangements for controlling the flow into the outlet-pipes, one or two sets of stop-cocks are often provided for each line of outlet-pipe. These stop-cocks are placed in a vault constructed near the toe of the outer slope of the dam (Plate LXIX.). The down-stream set of stop-cocks are generally used. If one of them should get out of order, the up-stream stop-cock of its line would serve to control the flow. The scour-pipe usually passes through this vault, where it is also controlled by a stop-cock.

Outlet Fountain.—When the outlet-pipes discharge into a stream the water is sometimes aerated by being thrown upwards in vertical jets in a fountain. If the pipes are of small diameter, their ends may simply be terminated by elbows which make them discharge vertically. In the case of large outlet-pipes, several smaller jets are substituted by pipe-connections for one large one. The basin of the fountain must have a sufficient capacity to contain the outflow from the jets without overflowing. At the point where the water flows from the basin, a weir may be constructed for measuring the water drawn from the reservoir.

Construction.—In selecting a site for a dam, careful investigations of the ground on the proposed location must be made by means of test-borings and pits. The core-wall or puddle-core of the dam must be founded on an impervious stratum (solid rock, hard-pan, clay, etc.). It is important to find a site for the dam where an impervious stratum can be reached at a reasonable expense. This stratum must have sufficient depth to be able to support the weight of the dam. Occasionally a thin stratum of impervious hard-pan or clay overlies very pervious material. The test-borings must, therefore, be carried to a sufficient depth to give a clear indication of the ground upon which the dam is to be founded.

Having selected a suitable location for the dam, the first step in beginning the construction is to remove the surface soil from the whole space that is to be covered by the dam. This material is deposited in temporary mounds known as "spoil-banks" and reserved to cover those surfaces of the finished dam that are to be sodded. If the impervious stratum be near the surface, it will be advisable to remove all the material

overlying it within the dimensions of the dam. Should this involve too much expense, only a trench for the core-wall or puddle-core need be excavated to the impervious stratum. To attain the desired object, this trench may have to be excavated to a great depth.

Springs are frequently encountered in making the excavations for a dam, especially in the puddle-trench. Whenever possible, they should be stopped by means of hydraulic mortar or masonry. If this cannot be done, the springs should be led in pipes beyond the toe of the dam, or they may be confined in vertical pipes which are finally closed by filling them with grout or clay.

If the core of the dam is to consist of masonry, it should be constructed either of concrete or rubble, laid in hydraulic cement mortar and made as water-tight as possible, its up-stream face being plastered with mortar.

The foundation of the core-wall may change from rock to impervious earth (hard-pan, etc.). If the wall has considerable height, it is apt to crack at the points where the foundation changes, owing to differences in settling. In such a case it is advisable to construct wells in the core-wall at the points of the above changes. which will permit an inspection and repairs of the wall if required. The wells may be filled at first with gravel and finally with masonry. A well of this kind was constructed in the core-wall of the Titicus Dam, described on page 93.

The general manner of forming a puddle-core has already been explained. In excavating the puddle-trench it is generally stepped where sudden changes of grade occur. If the steps have considerable height, they may cause cracks in the puddle-core on account of differences in settling. In such cases it is better to use inclines instead of steps, as this arrangement tends to consolidate the puddle towards the centre of the valley.

A core-wall may be carried up independently of the rest of the dam, but a puddle-core should be brought up simultaneously with the other portions of the dam.

The dam should be constructed in layers which are either horizontal or incline slightly towards the central core-wall or puddle-core. Some engineers make the layers 2 to 3 feet high, but it is preferable to give them a height of only 6 inches. After the earth has been carefully and uniformly deposited in a layer it should be thoroughly rolled, parallel with the axis of the dam, with rollers weighing about 150 to 300 pounds per lineal inch. The best rollers are grooved. In ordinary material the rollers pass at least six times over every portion of each layer.

During the rolling the earth must be sufficiently moistened by sprinkling to make it pack well, but only a little water must be used—there is danger in using too much. The wetting must never be more than a sprinkling. If the material is moist, no water will be required.

After the dam has been constructed, the top and outer slope, which are to be sodded, or seeded with grass-seed, are covered with about 6 inches of good top-soil or loam. The sods used should be of good earth covered with heavy, healthy grass, and should have a uniform thickness of about 3 inches. Each sod should be at least a foot square. The sides of the sods should be bevelled so that their edges should lap. The sods should be well bedded and padded down with a spade. On slopes a

sufficient number of sods must be secured to the ground by wooden pins, about 15 inches long, to keep the sodding in place. In dry weather the new sodding should be occasionally sprinkled.

Plates LXXIX., LXXX., and LXXXI. show earthen dams that have actually been constructed, except Fig. 1, Plate LXXIX., which gives the contract drawing for the earthen part of the New Croton Dam which is now (1899) being constructed. For the reasons explained on page 106, this dam, while built according to the profile given in Fig. 1, will have a maximum height of only about 80 feet. Figs. 2 and 3 of this plate show smaller earth dams which have recently been constructed for storage reservoirs for the city of New York.

In Plate LXXX., which we take from "The Construction of Catch-water Reservoirs," by Charles H. Beloe, a cross-section is given of one of the earthen dams which forms the New Yarrow Reservoir at the Rivington Water-works of the city of Liverpool. The dam has a maximum height of 90 feet. The reservoir, which is formed by two earthen dams, covers 73 acres and stores about 1,000,000,000 gallons.

Plate LXXXI., which we reproduce from William Humber's "Comprehensive Treatise on the Water-supply of Cities and Towns," shows an earthen dam with an outlet-tower, which was built for the Bombay Water-works. The dam forms the Vehar Reservoir, which covers 1394 acres and stores 10,800,000,000 gallons. The dam has a maximum height of 84 feet.

The outlet-tower has four inlets which are placed at intervals of 16 feet. The inlets are 41 inches in diameter and are provided with conical plug-seats, faced with gun-metal. A wrought-iron straining-cage covered with No. 30 gauze copper wire is placed over the inlet opening that is in use. A similar straining-cage (but made with No. 40 gauze copper wire) is placed at the bottom of the well, over the orifice of the supply-pipe. The tower is a good example of Indian architecture.

Failures of Earthen Dams have been very numerous. The cause of the rupture has generally been a neglect of some detail in the construction,—as an insufficient length of spillway or of the waste channel below it, a faulty manner of laying the outlet-pipes in the dam, etc. The two greatest disasters resulting from the failure of earthen dams occurred in Sheffield, England, on March 11, 1864, and in Johnstown, Pennsylvania, on May 31, 1889. As they teach some important lessons, we shall describe briefly the facts connected with these failures.

The Dale Dyke Dam formed the Bradfield reservoir for the water-supply of Sheffield. This reservoir covered 78 acres and stored 114,000,000 cubic feet. The dam was 95 feet high, 1254 feet long, 12 feet wide on top, and 500 feet at the base. Both slopes were $2\frac{1}{2}$ to 1. The puddle-core was 4 feet wide on top and 16 feet at the surface, both faces being battered $1\frac{1}{2}$ inches per foot. To reach an impermeable stratum the puddle-trench was excavated for a great part of its length to a depth of 60 feet. Two 18-inch socket-jointed cast-iron outlet-pipes ($1\frac{1}{4}$ inches thick) were laid naked in a trench under the dam at its highest point. The pipes were placed 2 feet 6 inches apart. The whole trench was refilled with puddle, 18 inches of this material being placed both below and above the pipes. Where this trench crossed that of the puddle-core of the dam, it was excavated to the depth of the latter.

In constructing the dam, the engineers adopted the rather original plan of making the inner part of the embankment as much as possible of rubble-stone and shale.* They based this preference on the idea that earth becomes saturated by water and assumes a flatter slope, while a pervious bank, made principally of stone, will keep its slope. By this arrangement, however, the whole hydrostatic pressure of the water was brought directly against the puddle-core. If settling caused the least crack in this puddle, the water was sure to find its way rapidly through the dam. While the puddle-core is to act as a cut-off in the heart of the bank, an inner slope of well-packed earth should prevent the water from percolating to the centre of the dam, as much as possible.

The reservoir was full when the dam failed, and a narrow crack had appeared on the outer slope. In the investigation which followed the rupture of the dam, the greatest engineers of England testified as experts. Their opinions, as regards the cause of the bursting of the dam, varied very much, and it will never be known what started the failure. It is evident, however, that the plans of the dam were very unsafe in requiring the outlet-pipes to be laid unprotected in the dam, and the inner part of the dam to be made of stone and shale.

The Johnstown Disaster, which caused the loss of more than two thousand lives and of millions of dollars' worth of property, resulted from the rupture of an earthen dam which was built across the south branch of the Little Conemaugh River in Pennsylvania. The dam was 70 feet high, and 10 feet wide on top. The inner and outer slopes were respectively 2 to 1 and $1\frac{1}{2}$ to 1. In this case the inner slope was made of earth properly rolled, but stone was placed in the outer slope. The failure, which occurred after an unprecedented rain-storm, was due to the insufficiency of the waste-weir, which was partly obstructed by fish-screens. At 11.30 A.M. the water commenced to pass over the top of the dam, and it rose to a height of 20 inches above the dam. The water gradually cut a channel through the embankment, until at 3 P.M. the dam burst.

If this dam had been provided with a masonry core-wall, carried up to the high-water mark, the water, instead of cutting a channel through the bank, would have washed away the earth to the top of the core-wall. This wall would have formed a long waste-weir which would not have been ruptured until the outer slope was washed away, and even then only the highest part of the wall might have given way. Considering the fact that the outer slope of the dam was made largely of stone, it is quite probable that the Johnstown disaster would not have occurred if the dam had had a core-wall. Such a wall, besides making a dam water-tight, may be considered as a safeguard against the erosive action of water that may pass over the top of a dam during a great flood.

*See "The Designing and Construction of Storage Reservoirs," by Arthur Jacob, B.A.

CHAPTER II.

ROCK-FILL DAMS.

WITHIN recent years a new style of dam has come into use in the Western States of the Union. We refer to what is known as a rock-fill dam, an embankment consisting of rock dumped loosely except at the faces, where it is laid carefully as dry slope-walls. Water-tightness is insured by a sheeting of boards or a facing of concrete on the water-slope, or by building an earthen dam against the inner or outer slope.

Rock-fill dams were first introduced for storing water for placer-mining, and have since been used for impounding water for irrigation. In mountainous regions, where the cost of transportation limits the use of cement, rock-fill dams will cost less than masonry dams. If placed upon an unyielding foundation (rock or hard-pan) and properly constructed, such a dam has ample strength. It will not be as tight as a dam of masonry, but the leakage can cause no damage. Where cement can be obtained at a reasonable cost, a masonry dam will generally be found to be cheaper than a rock-fill, as it has a much smaller cross-section. Under ordinary circumstances a rock-fill dam would be more expensive to construct than one of earth. Local circumstances may, however, change these conditions, and the fact that several high rock-fill dams have been built of late in the mountains of Western States leads to the inference that the engineers who designed the works found this style of dam to be the cheapest they could construct.

The various ways in which rock-fill dams can be built will be best illustrated by describing the construction of a number of such works. We have obtained most of the information given about these dams, and some others mentioned in Chapter III, made by the hydraulic process, from the very valuable report on "Reservoirs for Irrigation" by Mr. James D. Schuyler, M. Am. Soc. C. E., which was published in Part IV of the Eighteenth Annual Report (1896–97) of the United States Geological Survey. Some information on the subject is also given by Mr. Herbert M. Wilson, C.E., in his "Manual of Irrigation Engineering." Short accounts of some of these dams have also appeared in the engineering papers.

The Escondido Dam was the first rock-fill dam built in California to form a reservoir for irrigation purposes. It is 76 feet high, 140 feet wide at the base, and 10 feet on top. The dam is 380 feet long on its crest and 100 feet long at the river-bed. The slope on the water side is ½ to 1. On the other face the slope is 1 to 1 for the upper half and 1½ to 1 for the lower half. The dam consists of a fill of loose rock in large blocks weighing up to 4 tons. No quarry-spalls or earth were used in the fill. On the inner face the stones were carefully laid by hand to form a dry wall which is 15 feet thick at the bottom and 5 feet on top. The dam contains 37,159 cubic yards of rock, of which 6000 cubic yards were laid as dry wall. All the

stone used was obtained from boulders or outcropping ledges of rock, as no good quarry could be found near the work. It was transported on tramways and dumped from cars into the rock-fill, being only roughly placed by means of derricks. A trestle was built on the longitudinal centre-line of the dam to support the tramway. It was raised as the work required, the posts being left in the fill.

Before the dam was begun the whole space it was to occupy was stripped of soil, rock being uncovered at a depth of about 4 feet below the river-bed, extending almost level across the valley. As the bed-rock was found to consist of disintegrated granite holding boulders, a trench was excavated at the upper toe of the dam 3 to 12 feet deep into the bed-rock. A wall 5 feet thick, made of rubble laid in cement mortar, was built in this trench to prevent leakage under the dam and to serve as the foundation of a facing of planking which was placed against the up-stream face. Redwood timbers, 6 × 6 inches, were placed vertically in the dry rubble wall of this face, 5 feet 4 inches apart. They were embedded in the rubble 4 inches deep and projected 2 inches. Horizontal planks were spiked to these timbers, the 2-inch space between the dry wall and the planks being filled with concrete as each row of planks was laid. The planks used for the lower, middle, and upper third of the slope were respectively 3, 2, and 1½ inches thick. A second layer of planks of the same size was placed over the first one, the joints being broken as much as possible. The joints were calked and smeared with asphaltum. The facing of plank was carried up 3 feet above the top of the dam.

Springs were encountered in the foundation-trench. They were led in pipes to the outer toe. When the water was raised in the reservoir to the 57-foot level, the leakage from the reservoir was found to be about 100,000 gallons in 24 hours. It is doubtful whether this water percolated under the dam or leaked through the facing. The leakage has remained quite constant.

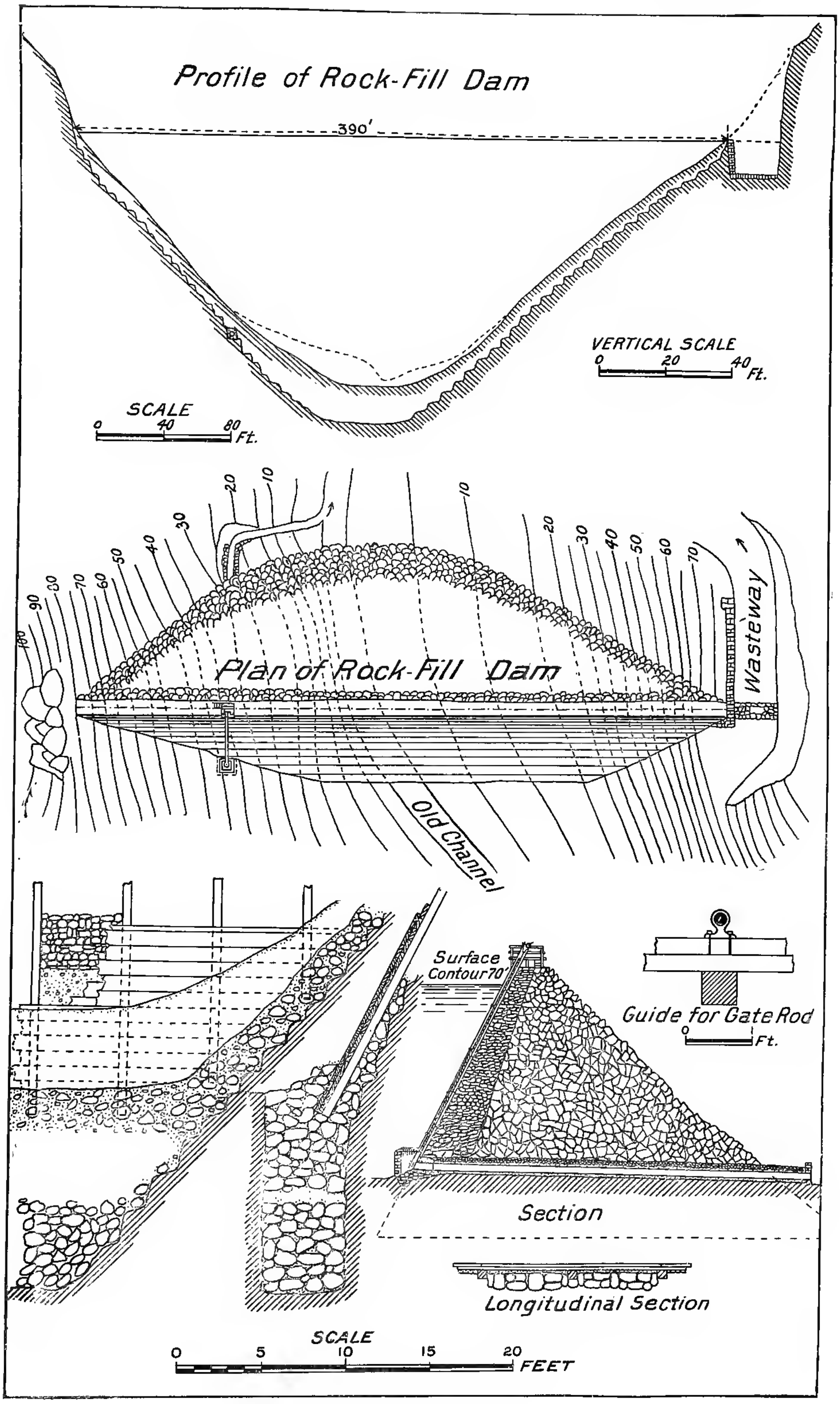
Water is drawn from the reservoir through a 24-inch vitrified pipe which was embedded in concrete in a trench passing under the dam. It was covered with 12 inches of concrete. The outlet is controlled by a gate which is set on the inner slope and operated by means of a rod leading to a worm-gear placed on top of the dam.

A spillway, 25 feet wide, was excavated in solid rock at the north end of the dam.

The cost of the rock-fill dam, not including land, amounted to $86,946.21, which is about $27.82 per acre-foot of reservoir capacity up to the flow of the spillway.

Walnut Grove Dam.*—This rock-fill dam was constructed across the Hassayampa River, about 30 miles from Prescott, Arizona, to impound water for irrigation and for furnishing power for working extensive gold placer-beds. The reservoir covered about 1000 acres and stored about 3,000,000,000 cubic feet. The dam had a height of 110 feet, and was 400 feet long on top. It was 15 feet wide on top and 140 feet at the base, the water-slope being 0.5 to 1, and the outer slope 0.6 to 1. The entire base of the dam for a height of 10 feet was made of rubble masonry laid in cement mortar. Above the base the dam was made as a rock-fill, with granite stone quarried

*Engineering News for 1888.



ESCONDIDO DAM.

(From "Eighteenth Annual Report of U. S. Geological Survey.")

near by and dumped from cars that were carried across the valley on a timber trestle built on the longitudinal axis of the dam. The trestle was raised as the work progressed, the posts being left in the fill. At the slopes, the stone was laid by hand so as to form dry face-walls.

Water-tightness was obtained by a facing of planking. Cedar logs were embedded vertically in the inner face-wall, about 6 feet apart. Longitudinal timbers, 8 × 8 inches, were notched and bolted to the cedar logs, about 3 feet apart. A sheathing of 3 × 8-inch planks, placed vertically, was spiked to the longitudinal timbers. This sheathing was covered with prepared tar-paper, about $\frac{1}{8}$ inch thick, which was secured by a horizontal sheathing of 3 × 8-inch plank. All the joints of the planks were carefully calked. The outer sheathing was coated with pitch and then with paraffine paint.

The outlet gate-house was built of timber and is 6 feet square in section. It was provided with gates for controlling two 20-inch outlet-pipes that pass through the base of the dam and were embedded in masonry.

A waste-weir, 6 by 26 feet, was blasted out of the rock on one side of the dam.

About 50,000 cubic yards of stone were required to make the dam. In addition to this about 12,000 cubic yards more from the waste-weir and channel were dumped in front of the dam.

The plans for the work were made by Prof. Wm. P. Blake, the well-known mining expert, but the execution of the work was left principally in the hands of the contractors and projector of the enterprise. The dam leaked considerably when the reservoir was first filled, but gradually became more water-tight. In February, 1890, however, the dam was completely destroyed during a great flood. This failure is ascribed to the insufficiency of the wasteway, which could not discharge the flood-water, and to carelessness in the execution of the work.

Lower Otay Dam.—This rock-fill dam was constructed on Otay Creek, about 20 miles southeast of San Diego, California, to form a reservoir for irrigation purposes and also for furnishing a domestic supply for Coronada Beach. The dam, which is 130 feet high, consists simply of a loose rock-fill, none of the stones being placed by hand. It is 20 feet wide on top, and both slopes are made 1 to 1. Water-tightness is insured by placing a core of steel plates in the centre of the fill, forming a web-plate across the canyon.

The original plans for the reservoir contemplated the construction of a masonry dam. The foundation for the wall was actually laid 63 feet wide and carried up about 40 feet high. This block of masonry was used as the foundation for the steel web, which was placed 6 feet from the up-stream face of the masonry. The bottom plates were 5 feet wide and 17.5 feet long. Above a height of 50 feet plates 8 feet wide by 20 feet long were used. In the lower three courses the plates are 0.33 inch thick. All the others have a thickness of $\frac{1}{4}$ inch.

The plates were riveted together in position and calked. They were coated with hot asphalt, and covered on both sides with burlap which had been saturated with asphalt. To stiffen and protect the plate against the rock dumped around it, a masonry wall was built on each side against the plate. Each of these walls is 6 feet thick

at the base and tapers to a thickness of 1 foot at a height of 8 feet, above which the walls are uniformly 1 foot thick. At the sides of the valley the plates were placed in trenches cut in the solid rock, securely anchored and protected by masonry. As might have been imagined, considerable difficulty was experienced in keeping the plates in line on account of expansion and contraction due to changes in temperature.

The stone required for the dam was all quarried below the work, brought to the fill by a Lidgerwood cableway, and distributed by means of derricks. The largest stones were placed on the down-stream side of the plate-core, smaller stones and earth being used on the up-stream side. The dam contains about 140,000 cubic yards of rock.

No pipes pass through the dam. The outlet is made through a tunnel 150 feet long. For the first 500 feet from the reservoir the tunnel is lined with 12 to 18 inches of concrete, so as to form a circular conduit having an inner diameter of 5 feet. At the end of this conduit a shaft 104 feet high reaches the surface, and serves for operating, by means of rods, a sluice-gate, which controls the outlet. Beyond the shaft a 48-inch steel pipe was placed in the tunnel and surrounded with about 1 foot of concrete, collars being carried to the side of the tunnel every 25 feet.

The overflow was made in a depression several hundred feet away from the dam. The watershed above the reservoir contains about 100 square miles.

The Chatsworth Park Dam was constructed in 1895–1896, in the San Fernando Valley, California, to impound water for irrigation. It was a rock-fill dam about 41

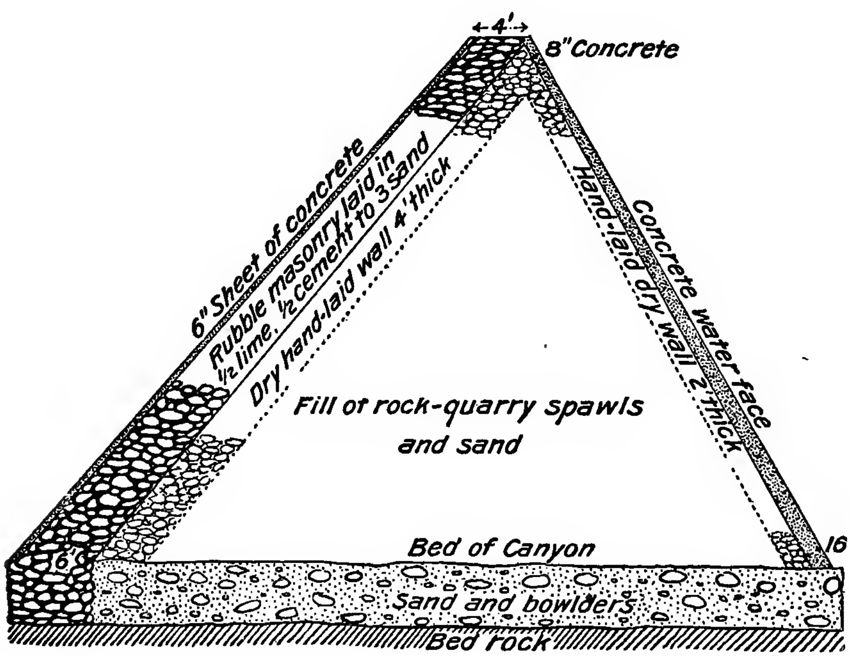


FIG. 19.—SKETCH OF RECONSTRUCTION OF CHATSWORTH PARK ROCK-FILL DAM.

feet high, 10 feet wide on top, both slopes being at an angle of 60° (1 vertical in 0.57 horizontal). The length was 159 feet on top and 100 feet at the bottom. On the slopes, the stones were laid to form dry walls, 2 feet thick. The slope wall on

the water side, which contained 7700 square feet, was covered with Portland-cement concrete 8 to 16 inches thick.

The dam was made with soft sandstone which was quarried near by. The work appears to have been executed in a careless manner. With a depth of only 10 feet of water in the reservoir the dam leaked so badly that the company for which it was constructed decided to rebuild the dam. The plan adopted for the new dam is shown in Fig. 19 on page 128.

Pecos Valley Dams, New Mexico.—Two rock-fill dams, differing from those already described, were built in the Pecos Valley, about 15 miles above the town of Eddy. In these dams leakage was stopped very successfully by forming the down-stream part of the dam of earth.

Fig. 20 shows a section of the lower dam which has a length of 1380 feet. The dam was built in 1889–1890. Owing to the insufficiency of the spillway water flowed

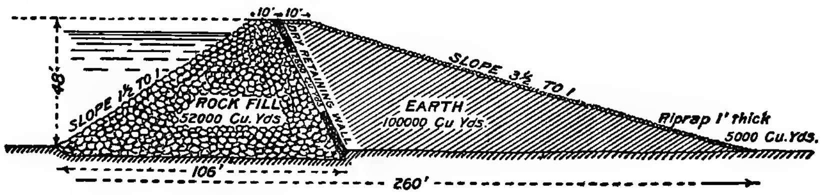


FIG. 20.—PECOS VALLEY DAM. NO. 1.

over the top of the dam, in August, 1893, and washed out a breach of over 300 feet. The damage done was repaired and the dam was raised 5 feet higher. Additional spillway was also provided.

The upper dam (Fig. 21), which is 1686 feet long on top, was constructed in 1893 like the lower one, excepting that the rock-fill part was made 4 feet wider on top and

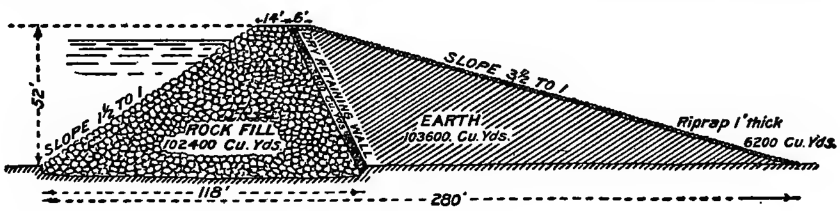


FIG. 21.—PECOS VALLEY DAM. NO. 2.

the earthen part 4 feet less, the whole top-width remaining 20 feet. The inner slope of the rock-fill, against which the earth bears, was laid up by hand as a dry stone wall 2 feet thick. The dam cost $170,000.

The type of rock-fill dam described above appears to be perfectly safe if an ample waste-weir be provided.

The Idaho Dam* differs from the Pecos Valley dams in having the facing of earth placed on the up-stream side of the rock-fill. This dam was built across the Boise River at the head of the Idaho Mining and Irrigation Company's canal. It is 220 feet long on top and 43 feet high, and is made of loose rock except the facing

* "Manual of Irrigation Engineering," by Herbert M. Wilson, C.E.

of earth, which is 3 feet thick at the top of the dam and 20 feet thick at the bottom. The dam is 10 feet wide on top. The outer rock-slope and inner earth-slope are both $1\frac{1}{2}$ to 1.

The Castlewood Dam,* Fig. 22, was built across Cherry Creek, about 35 miles south of Denver, Colorado, to store water for irrigation. This stream is liable to great changes of flow. Ordinarily its volume is very small, but during sudden freshets, following so-called cloud-bursts, it discharges as much as 10,000 cubic feet per second.

The dam was begun in December, 1889, and completed in November, 1890. It is 600 feet long on top and 8 feet wide. Its maximum height is about 70 feet above the surface and 92 feet above the foundation. This dam differs from all the other rock-fill dams we have described, in having a facing of rubble laid in cement mortar at its inner and outer slopes. Between the face-walls the dam consists entirely of loose stone dumped on the natural surface.

The inner face-wall is 4 feet thick and is carried up on a batter of 1 foot horizontal to 10 feet vertical, except at the highest part of the wall at the overflow, which is placed in the middle of the dam, where the inner wall for a stretch of 120 feet was made vertical on the side next to the rock-fill. The foundation-trench for the inner wall was excavated in an arenaceous clay, containing large boulders, to a depth of 6 to 22 feet.

The outer face-wall is carried up in steps on a general slope of 1 to 1. Its foundation is nowhere more than 10 feet below the surface. The steps were formed of dimension-stone, which extend at least 3 feet into the dam. Both the inner and outer face-walls rest on footing courses of concrete 1 to 2 feet thick.

The face-walls are carried up on the slopes mentioned to the elevation of the spillway, where they are united. The top of the dam is formed of a wall of rubble masonry, 8 feet wide and 4 feet high, having vertical faces.

The overflow, which is placed in the middle of the dam, is 100 feet long by 4 feet deep. In addition to this a by-pass 40 feet wide is provided on the west side of the dam, and aids in discharging the surplus water. Its floor and sides are lined with masonry to a safe point of discharge.

The outlet-well is built in the centre of the dam adjoining the overflow-weir. It measures, on the inside, $6 \times 7\frac{1}{2}$ feet. The walls of the well are 4 feet thick, except towards the reservoir, where the thickness of the wall is increased by offsets to 10 feet at the surface. As these offsets are made on the inside of the well, to provide a foundation for the valves controlling the inlet-pipes, the opposite wall is recessed out to maintain the inner dimension of the well. Where the recesses are made, the wall is supported by arches. Eight 12-inch inlet-pipes, placed in pairs at four different elevations, admit the water to the well, from which it is conveyed by a 36-inch concrete conduit and discharged into the creek a short distance below the dam.

The Castlewood Reservoir covers 200 acres of land and stores about 4,000,000,000 U. S. gallons. It was built for the Denver Land and Water Company according to the plans prepared by their Chief Engineer, Mr. A. M. Welles. The plans contemplated

* Engineering Record, Dec. 24, 1898, and Engineering News, Feb. 9, 1899.

placing an earthen slope against the inner face of the dam to a certain height. This was not done, however, until later, after the dam commenced to leak badly. The contract for the work was given to the Rosenfeld Construction Co., who employed their own

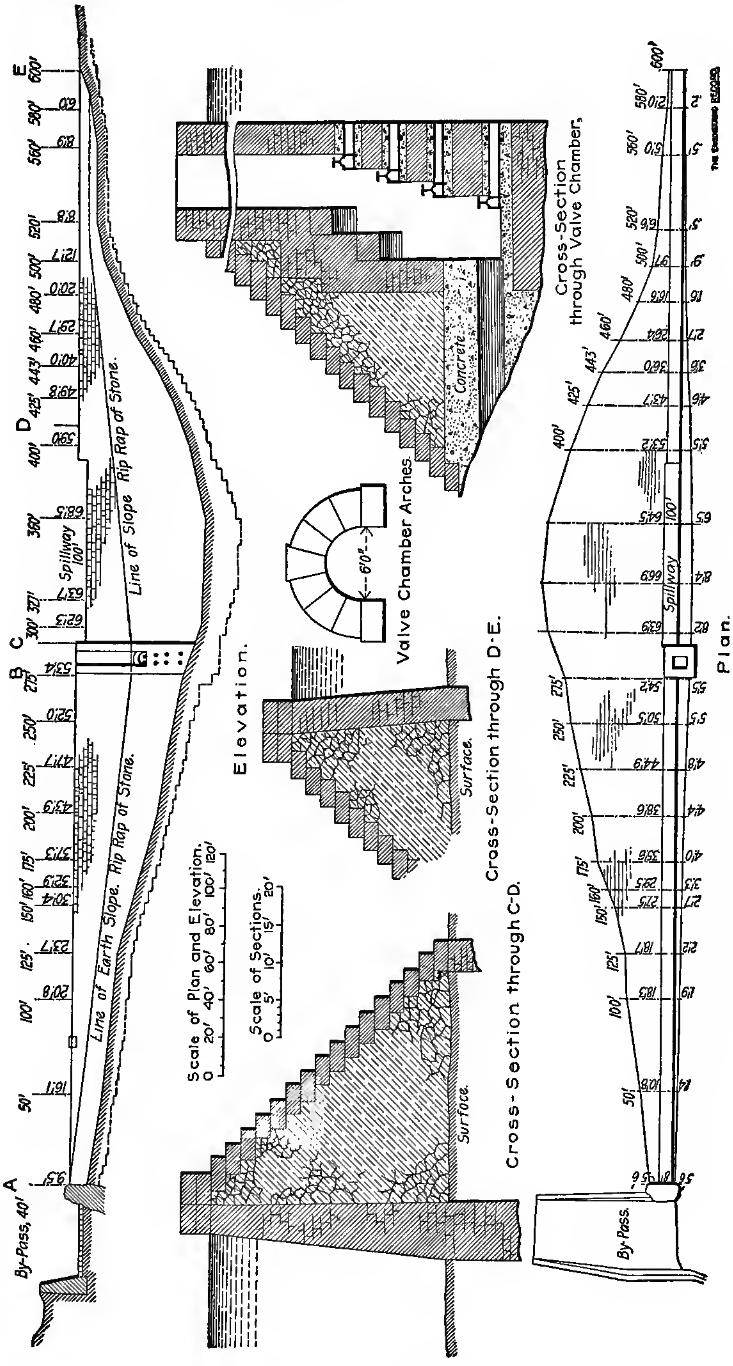


FIG. 22.—THE CASTLEWOOD DAM. (By permission of "The Engineering Record.")

engineers. Although the Denver Land and Water Co. had inspectors on the ground, the work appears not to have been well executed in some particulars. Settling occurred at some points, causing cracks 2 to 4 inches deep, through which the water found an outlet. At one point the water appears, also, to have flowed under the dam.

The dam was repaired and an earthen slope placed at its up-stream face The top of this bank is 35 feet below the crest of the dam at the centre and rises gradually so as to reach the crest at both ends of the dam. This slope is covered by a rip-rap 1 foot thick. Where the leakage occurred clay puddle was placed next to the inner wall.

The plans of the Castlewood Dam have been severely criticised, especially the fact that the inner face-wall, where only 4 feet thick, overhangs the loose rock-fill, its centre of gravity falling outside its base. Eight years of service have, however, not yet shown any disadvantage resulting from this feature. Engineers will be interested in watching the future history of this dam.

CHAPTER III.

DAMS MADE BY THE HYDRAULIC PROCESS.

A NOVEL manner of building dams of earth and gravel has been used in some of the Western States. It consists in excavating, transporting, and depositing the material required by the erosive action of water which is obtained either under pressure from a jet or by gravity from a flume. This method, which was first introduced for what is known as "hydraulic mining," was soon applied to making small dams in California. The dam of Temescal and San Leandro storage reservoirs for the water-supply of Oakland, California, were formed in this manner. The hydraulic process has also been used on a very extensive scale in making embankments for the Northern Pacific and Canadian Pacific Railways.

The Temescal Dam was built in 1868. It is 105 feet high, 18 feet wide on top, and had both sides sloped $2\frac{1}{2}$ to 1. These slopes have been since flattened by material that was sluiced in from year to year. The reservoir formed has an area of 18.5 acres and stores 188,000,000 gallons.

The San Leandro Dam, which is 120 feet high, was built in 1874–1875. The dam contains 542,700 cubic yards of material, of which 160,000 cubic yards was deposited by the hydraulic process. The water used was brought for a distance of 4 miles in a ditch. The material was sluiced in a sheet-iron flume, laid on a grade of 4 to 6 per cent. As it was not convenient to get water under pressure, the ground-sluicing method was used. It was estimated that the material placed by the hydraulic process cost only about one quarter to one fifth of that placed with carts or scrapers.

The Dam at Tyler, Texas, was built in 1894 by the hydraulic process. The dam is 575 feet long and 32 feet high. The inner and outer slopes are respectively 3 to 1 and 2 to 1. The maximum depth of the water in the reservoir is 26 feet. All the material used in the dam was sluiced in from a hill near by. The average cost of the dam, including the plant and all the appurtenances of the reservoir, was $4\frac{3}{4}$ cents per cubic yard.

In this case the water required for sluicing was obtained from a 6-inch pipe from the old city pumping-station. This pipe terminated with a common fire-hydrant about half-way up the hill from which the earth was to be washed. An ordinary $2\frac{1}{2}$-inch hose, with a nozzle $1\frac{1}{2}$ inches in diameter, was connected with the hydrant and delivered the water, at the place where it was required, under a pressure of 100 pounds per square inch. The stream from the nozzle was directed against the face of the hill. The cutting made by the jet was carried into the hill on a 3 per cent grade. A working-face of 10 feet high was soon obtained and gradually increased to 36 feet. The jet was directed against the face so as to undermine it, and the water washed the material

(clay, sand, and loam) to the dam. About 65 per cent of this material was sand, and 35 per cent clay and loam.

The work on the dam was begun by digging a trench 4 feet wide from the surface down into the clay subsoil, a depth of several feet. This trench was then filled with selected puddle-clay which was sluiced into place. The slopes of the dam were then defined by low ridges made by the laborers with hoes, and a flow of water carrying sand and clay was maintained over the top of the dam, the water being drawn off from time to time at either slope. The material was conveyed from the bank in a 13-inch sheet-iron pipe having loose joints, stove-pipe fashion. This pipe extended from the bank to and across the dam on its centre-line. The joints could be readily uncoupled and the stream directed so as to carry the bank up uniformly. The quantity of solids brought down by the water was found to vary from 18 per cent in clay to 30 per cent in sand. As sharp sand does not flow as readily as rounded sand or gravel, the delivery was increased by mixing clay and stones with the sand.

The entire cost of this dam is given as $1140. The reservoir formed by it covers 17.7 acres and stores about 77,000,000 gallons. The dam is reported to be water-tight.

La Mesa Dam, California, was constructed in 1895 to store the flood-water of San Diego River. The dam (Fig. 23) is 66 feet high, 20 feet wide on top, and

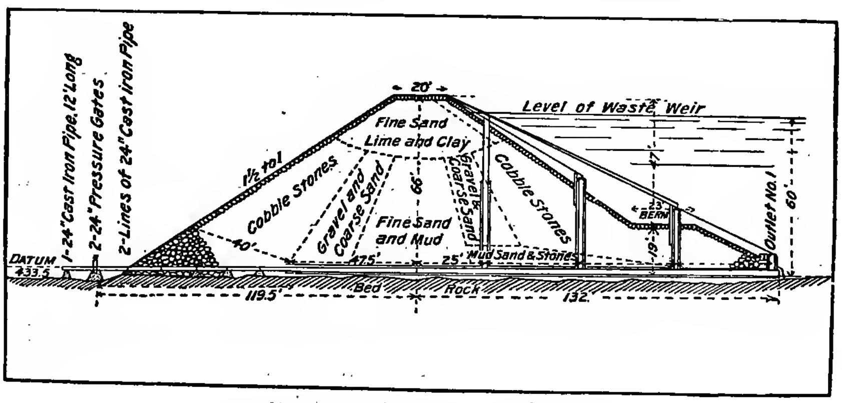


FIG. 23.—CROSS-SECTION OF LA MESA DAM.

251.5 feet at the base. It consists partly of a rock-fill and partly of a bank of earth. The material was transported and deposited in the fill by water by the process known to miners as "ground-sluicing." The surplus water from a flume of the San Diego Flume Company was used for the purpose and was stored in the reservoir as the fill rose in height.

The dam is located in a very narrow gorge cut through porphyry. The valley is only 40 feet wide at the bottom, and one side is almost vertical for 40 feet. According to the original plans a rock-fill with plank-facing was to be built at this site. This dam was to be 55 feet high. Its top-width was to be 12 feet, and the up-stream and down-stream slopes were to be respectively ½ to 1 and 1 to 1. The length on top was to be 470 feet. The lowest bid received for building this dam, exclusive of the plank facing, outlet-pipes, and gates, was $20,260. As a lower bid was received for building a dam by the hydraulic process, that method was adopted and the dam was

PLATE J.

Beginning the Construction of La Mesa Dam.
(From "Eighteenth Annual Report of U. S. Geological Survey.")

built at a total cost of about $17,000, including plant, excavation of foundations and spillway, outlet-pipes, culvert and stand-pipes, paving of slopes, etc.

About 38,000 cubic yards of material was put in the dam. It was obtained by stripping 11.5 acres, situated about 2200 feet from the dam, a mean depth of 2 feet. Below the depth of 2 feet the material was found to consist of gravel and cobbles, which were cemented together so hard as to resist washing. This necessitated the use of scrapers to bring the material to the sluiceway and increased the cost considerably. If all the material could have been transported directly by the hydraulic process, the dam would probably have cost 25 to 30 per cent less.

The gravel put in the dam varies from egg size to cobbles 8 to 10 inches in diameter. The largest cobbles were laid by hand on the outer slope so as to form a dry wall of uniform batter.

The amount of water available for building the dam was only from 300 to 400 miner's inches (6 to 8 second-feet). From the end of the flume from which the supply was obtained, the water was siphoned across a deep ravine in a 36-inch wooden-stave pipe, 3000 feet long, which emptied into a ditch 1.5 miles long, extending to the top of the ridge on the south side of the dam. Lateral ditches were carried from various points on the main ditch down the slope on 6 per cent grades. They divided the ground into irregular zones, 50 to 100 feet in width and several hundred feet long. These divisions were stripped to the rock, beginning next to the dam and working towards the ridge. The fall from the upper (clear-water) ditch to the lower side of a zone was made as great as possible. When the slope became flatter than 1 in 4, the velocity of the water was reduced so as to become insufficient to erode the material. In such cases the hydraulic process had to be assisted by the use of scrapers and ploughs, where the ground was not too soft for teams, or by hand labor.

The stream of water carrying its load of earth and gravel was conveyed along the line of the lower ditch through a 24-inch wooden-stave pipe to the fill where the material was to be deposited. This pipe was found to wear out very rapidly and had to be lined with strap-iron or tire-steel. Cast-iron pipes would have been preferable for this kind of work. An embankment made by this process becomes so thoroughly compacted that no rolling is required to prevent settling. In building this dam a force of 27 to 45 men, divided into three 8-hour shifts, placed 700 to 1400 cubic yards a day. Two men were always kept on the dump directing the stream of material, the other laborers being needed for the ground-sluicing. The upper 12 to 15 feet of the dam was finished by hauling in material with wagons.

Before beginning the dam a trench, 2 to 5 feet deep, 20 feet wide at the centre, and 5 feet wide at the ends, was excavated to the bed-rock on the longitudinal axis of the dam. The material excavated from the trench was thrown on both sides and forms part of the embankment.

Water is drawn from the reservoir through two lines of 24-inch cast-iron pipes, which extend through the dam at its widest part, for 72 feet from the outer toe, and connect with a concrete conduit (48 inches wide and 30 inches high) which connects with the reservoir. Four stand-pipes, consisting of 24-inch vitrified pipes, which are surrounded with concrete, connect with the conduit. Their tops are placed at different

levels. Each of these stand-pipes is provided on top with a brass ring and flap-valve, which is operated from the top of the dam by means of rods, laid on the inner slope. During the construction of the work these pipes served to admit the water into the reservoir after it had deposited its load of gravel and sediment. The pipes were carried up a joint (2 feet) at a time. As the stand-pipes are placed on the inner slope, the coarser material was deposited by the water in the outer slope, and the fine sand at the reservoir. This is just as it should be. An advantage of making a dam by the hydraulic process is that the work is tested, as it progresses, by the pond of water that collects behind it.

This dam is not free of leakage. With 46 feet of water the loss amounted to only 23 gallons per minute, but it increased to 100 gallons per minute when the water reached the 54-foot level. It is intended to cover the water-slope with a facing of asphaltum cement concrete.

CHAPTER IV.

TIMBER DAMS.

General Requirements.—Dams of brushwood, logs, cribwork, or framed timber are often built across streams to obtain water-power, to secure sufficient depth for slack-water navigation, or to divert water for irrigation. These dams are usually made strong enough to pass the streams over their crests in times of flood. They must also be able to withstand shocks from floating bodies such as ice, etc. This last requirement determines the form of the up-stream side of the dam, which should be an inclined plane (Fig. 24), in order to facilitate the passage of floating bodies and to protect the dam against shocks.

Dams have been built according to this simple profile, but unless the height of the dam be very inconsiderable the falling water will gradually undermine the dam in front, even if the bed of the river be rock and a pool of water protect it. Trautwine * states that at the Jones's Dam on Cape Fear River, which had a height of 16 feet, the water falling vertically over the dam, usually from a height of 10 feet, into a pool of water 6 feet deep, wore out the soft shale rock (in vertical strata) on which the dam was founded for 16 feet, and undermined the dam in a few years to such an extent that it fell into the cavity.

FIG. 24.

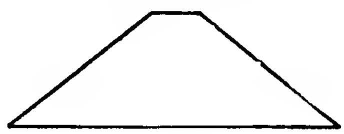


FIG. 25.

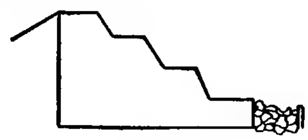


FIG. 26.

He mentions another case, where water falling vertically from a dam 36 feet high into a pool of water only 2 feet deep wore out the hard slate rock in the river 10 to 20 feet in twenty years, the erosive action extending from the face of the dam for a distance of from 70 to 80 feet.

The Holyoke Dam (page 147) is another example of the undermining of a dam by the erosive action of water flowing over it.

There are two ways of breaking the fall of the water flowing over the dam:

1st. By giving the down-stream side of the dam a slope (Fig. 25).

2d. By forming this face by a number of steps (Fig. 26). The latter plan is the better of the two, as the velocity of the water flowing down an incline plane is accelerated.

Whichever of the types shown in Figs. 24, 25, and 26 be adopted, an apron should be constructed in front of the dam to protect the river-bed against the erosive action of the overflowing water. The apron may consist simply of a layer of large stones and boulders. If the river be liable to severe freshets, the stones forming the apron must be kept in place by building a crib-dam or driving piles on the down-stream side of the layer. The whole

* Civil Engineers' Pocket Book, by John C. Trautwine.

apron may consist of crib-work filled with dry stones. Very frequently it is formed of a course of heavy timbers (10–14 inches thick) placed closely together.

The distance to which the apron should extend down-stream depends upon the height of the dam and the greatest depth of the sheet of water that may pass over it. Under ordinary circumstances the length of the apron, measured down-stream, should be about twice the height of the vertical fall of the water.

For dams founded on rock the apron is sometimes omitted, but this should only be done under the most favorable circumstances, viz., with hard rock covered by a pool of water having sufficient depth.

Timber dams may be built, in plan, either straight or curved so as to be convex up-stream. When a straight plan is adopted the axis of the dam may either be at right angles or oblique to the current. The latter plan is sometimes adopted with a view of obtaining a longer crest for the dam, but it has the disadvantage of tending to force the current towards the bank of the river on which the upper end of the dam abuts. Instead of curving the plan of the dam, it may be pointed up-stream, especially in the case of a narrow river.

Leakage may occur through the dam, around its ends, or under it. The up-stream face should be made as water-tight as possible. Some leakage will occur when the dam has just been completed. The silt and mud borne by the stream will soon diminish this loss. While it is important to prevent the water from leaking through the dam, it is rather an advantage, after the water has passed the crest of the dam, to have enough leakage take place to keep the timbers forming the structure always wet. For this purpose the planks which are used to cover the top and down-stream slope are often placed a little distance apart.

Leakage around the end of the dam is to be prevented by carrying the ends into the banks and building against them substantial abutments which should be raised to a height which will prevent their being overflowed. For an important dam the abutments should be built of solid masonry. A cheaper construction consists in forming the abutments of crib-work filled with stones or simply of sheet-piling.

The most dangerous leakage is that which may occur under the dam, as it would undermine the structure. The surest method of preventing this leakage is to give the dam considerable width up and down stream. As a general rule a timber dam should be wider than high. The greater the width can be made the better it will be.

In a river having a soft bottom one or two rows of sheet-piles (2–4 inches thick) should be driven at the up-stream toe of the dam. It is sometimes advisable to drive sheet-piles, also, at the down-stream end of the apron.

Having considered the general requirements which a timber dam built across a stream should fulfil, we will next consider the different manners in which wood can be used in the construction of a dam.*

Brushwood Dams.—In the case of a sluggish stream, having a soft bottom, a substantial dam can be formed of alternate courses of brushwood and gravel. Wood of all sizes, including saplings and even trees, should be used, the latter being always placed with their branches up-stream. After a course of brushwood 3 to 5 feet thick has been placed in position it is sunk by filling stone and gravelly earth upon it. Clay should be used but sparingly and with other earth, as it is apt to wash away.

* The descriptions of the simple types of dams given on pages 139 to 145 have been taken principally from Leffel's Construction of Mill-Dams.

The dam is carried up by alternate courses of wood and gravel so as to have a trapezoidal cross-section. It is finished by facing the slopes with planking, fascines, or a covering of riprap.

Log-dams.—Where timber can be obtained cheaply an excellent dam can be built, at a very moderate cost, of logs and brush, as shown in Fig. 27. The logs should be 8 to 12 inches in diameter at the butt end. The branches should be cut off the two sides of each log, which will be its top and bottom when placed in the dam. All the logs are placed with their tops up-stream.

The largest logs are laid side by side across the stream to form the foundation course. The second and third courses of logs are stepped up-stream respectively about 25

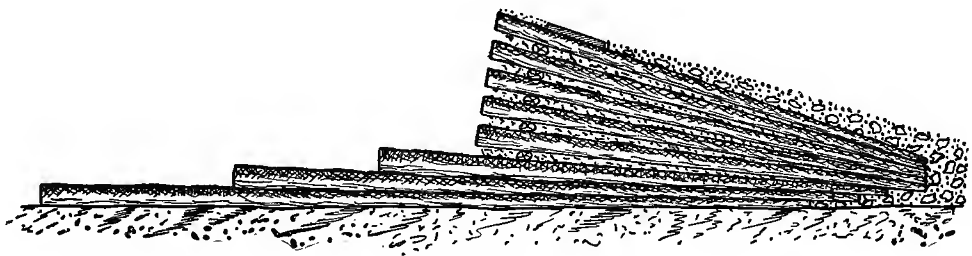


FIG. 27.

and 20 feet. The fourth course is stepped back about 5 feet from the third course, and the dam proper is then carried up with logs, so as to have its down-stream face almost vertical. Saplings, brush, stone, and earth are placed between the succeeding courses of logs to make the dam as tight as possible. Binders, 3 to 4 inches in diameter, should be placed across the logs of all the courses of the dam proper, and should be fastened by treenails or spikes to the logs. The top course should have several binders and should be covered with stone and earth so as to have a uniform slope.

A log-dam is especially suited for soft or sandy river bottoms. Owing to its great width and ample apron it will not be undermined. It will pass severe floods without damage, as the logs, brush, and filling are strongly interlocked. Experience shows that such a dam may settle one or two feet during the first year, but after that period the settling will be but trifling.

The log-dam may be straight or curved in plan according to circumstances. If much water flows in the stream while the dam is being built, the foundation courses of logs will have to be sunk by loading them with stone. An opening will also have to be left in the dam to pass the stream during the construction. It must finally be closed by building the dam at this place as rapidly and strongly as possible.

In the case of a narrow stream (about 40 to 60 feet wide) a strong dam can be built of logs by adopting the pointed plan. The butts of the logs are placed against the banks and the points are notched where they cross each other. There are only two logs in each course—one from each bank. The logs must be fastened together with treenails or drift-bolts. The dam is like a roof placed on end. Its strength depends evidently upon an unyielding bearing or skew-back being provided for the butt ends of the logs. If the banks of the stream be rocky, a good bearing is obtained by trimming the rock to the required surface. When the banks consist of gravel or earth, timber-cribs filled with stone may be placed in the

banks to form the skew-backs for the logs. In the latter case the logs forming the dam should extend into the crib, being notched and spiked to its timbers. Water-tightness is obtained by filling in a slope of gravel on the up-stream side of the dam. An apron of logs placed closely together, having their top and bottom sides squared, should be constructed on the down-stream side.

Dams 10 to 15 feet in height have been built according to the simple plan just described, and have stood successfully for many years.

Crib-dams.—A more economical plan of using logs to form a dam than the plan shown in Fig. 27 consists in building cribs with the logs and filling the spaces between them—ordinarily 6 by 6 feet to 10 by 10 feet—with stone or gravel. Fig. 28 shows a crib-

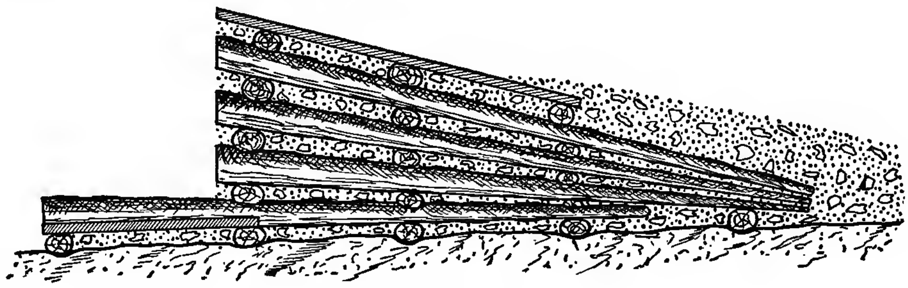


FIG. 28.

dam which has a triangular profile like the log-dam. The foundation course is formed of large logs, placed at right angles to the stream, and carried into the bank on both sides. These logs, which are generally placed 6 to 8 feet apart, are laid in trenches excavated to such a depth that the tops of the logs project just above the river-bed. If the width of the stream be considerable, two or more logs spliced together will be required for each of these trenches. The second course of logs is laid at right angles to the foundation course. The apron is formed between the two foundation-logs which are furthest down-stream by placing planks between the logs of the second course. These planks should project under the third course of logs, with which the dam proper begins. Each course of logs is placed at right angles to the one below it. The logs are not notched where they cross each other, but simply flattened so as to form good bearings. They are spiked together by iron drift-bolts (usually $\frac{3}{4}$ by $\frac{3}{4}$ in.) at each intersection of logs. Wooden tree-nails of hard wood may be used instead of the spikes. The top cross-logs should be securely fastened by iron bolts passing through two or three logs beneath. In building cribs the timbers should be so placed that the pockets of the crib will have vertical sides. Formerly the timbers were often staggered so as not to be directly over those below, but nothing is gained by adopting this plan.

In the dam shown in Fig. 28 the triangular section is obtained by making the logs across the river smaller at the up-stream than at the down-stream side of the crib. The down-stream face of the dam is made almost vertical. A course of planks (about 4 inches thick and 12 feet long) securely spiked to the logs is placed on the up-stream side of the crest of the dam. This course is continued up-stream by a slope of gravel or earth.

Crib-dams can be used in almost any kind of river bottom. If placed on rock the bottom logs should be fastened to the foundation by iron bolts. For this purpose holes are drilled in the rock. The lower end of each iron anchor-bolt is split from 5 to 6 inches.

By placing a wedge in the split end of the bolt and driving the latter down into the drill-hole, the bottom of the bolt is expanded and anchors the log firmly to the rock.

Unless the dam is to have but little height, it will be found most convenient to form it of square cribs, placing a low crib in front of the main dam to form the apron, and a slope of gravel and earth on the up-stream side. This plan is the method usually adopted. Descriptions of some dams of this kind which have been constructed and which have stood successfully for many years are given on pages 145 to 147.

Pile-dams.—If the river bottom is soft, but does not contain quicksand, a substantial dam can be built by driving one to three rows of piles across the river, the piles in each row being driven as closely together as possible. Logs and brushwood are placed horizontally between or against the piles, according to circumstances. In Fig. 29 two rows of

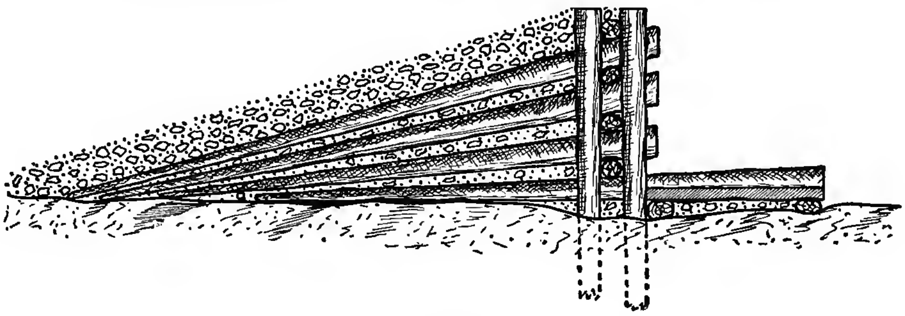


Fig. 29.

piles are shown, the horizontal logs and brush being placed between them. An apron is placed in front of the dam, and long piles are laid about 10 feet apart, as ties from the piling into the earth filling.

Plank-dams.—A strong dam can be formed by laying planks so as to form a vertical arch, convex up-stream. Planks 10 to 12 inches wide and 2 to $2\frac{1}{2}$ inches thick may be used for this purpose. They should not be more than 12 feet long, so as to form short chords of the arch. If the dam be founded on hard rock no apron is required. A level bed must be prepared for the first course. Concrete or rubble masonry may be used to level the irregularities of the rock. At both ends of the dam skew-backs must be cut for the arch to bear against.

Having laid the first three or four courses of planks, they should be anchored to the rock by iron split bolts in the manner described on page 140. The heads of the bolts must be countersunk in the top plank. The other courses of planks are merely spiked or fastened by treenails to those beneath them. The joints between the planks should be cut on radial lines and closely fitted. With this dam, too, it is advisable to place a slope of earth and gravel on its up-stream side.

If a dam is to be built of planks on a soft or sandy river-bed an apron must be provided. A foundation is prepared for the dam by laying long, square timbers about 10 by 12 inches in section, or logs squared roughly to this size, either closely together or 2 or 3 feet apart according to the softness of the bottom and the height of the dam. The down-stream part of these timbers forms the apron, if they are placed closely together. If they be two or three feet apart planks are spiked to them to form the apron. Instead of building a single arch of planks, as described above, a double arch may be built as shown in Fig. 30, the space between the arches being filled

with earth, gravel, and stone. Such a dam would of course be stronger than the single-arch dam described above.

As the dam shown in Fig. 30 is supposed to be built in soft ground, strong abutments must be constructed to resist the thrust of the arches. Cribs made of planks

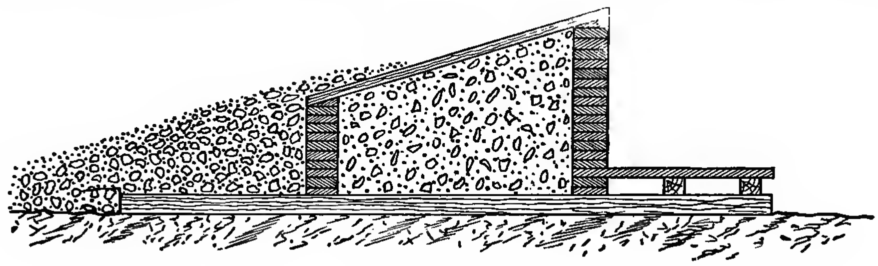


FIG. 30.

and filled with stone will answer for this purpose. Where the arches abut against the cribs the alternate courses of planks in the arches should be extended into the plank-cribs in order to tie the work well together.

A third plan of building a dam of planks is shown in Fig. 31. In this case the planks form a series of steps on either face, except at the lower part of the up-stream face, where they are laid as a vertical wall. Each course is securely tied by laying planks about 8 to 10 feet apart, at right angles across the dam, as shown in Fig. 31. The

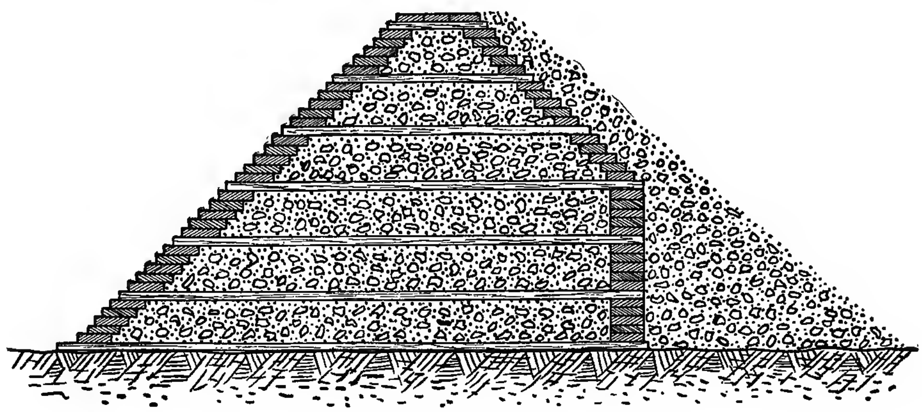


FIG. 31.

space between the planks is packed with earth and gravel and a slope of earth is placed on the up-stream side.

The planks used should be 10 to 12 inches wide and 2 to 3 inches thick. They may be spiked together, but it is preferable to fasten them together by wooden pins. These pins are usually made square to avoid splitting the planks in driving the pins. For soft-wood planks pins $\frac{3}{4}$ by $\frac{3}{4}$ inch are driven into round holes $\frac{3}{4}$ inch in diameter. If the wood be hard the holes must be made somewhat larger.

The planks on the down-stream side should be of oak. On the up-stream side planks of sycamore, elm, etc., may be used below the water, but above it the planks should be of oak.

If a dam of the kind just described is to be placed on a soft bottom, a foundation of logs or square timber must first be laid, as already explained.

Framed Timber Dams are made in various ways, according to circumstances. On a rock bottom a dam of moderate height can be built like a tight board fence. For a dam 6 feet high, posts 16 inches square are placed about 12 feet from centre to centre, the foot of each post being put in a hole about two feet deep excavated in the rock. Each post is supported on the down-stream side by an inclined brace 12 inches square. One end of the brace is let about a foot into the rock and bears against a piece of 2-inch planking. The other end bears against a shoulder cut in the post. A thin key serves to wedge the brace and post firmly together.

Three horizontal timbers, about 6 by 10 inches, are placed in notches cut in the posts, on their up-stream side. One of these timbers is securely spiked at the bottom, one at the middle, and the other at the top of the post. Vertical pieces of 2-inch plank are nailed to the horizontal timbers. If well-seasoned plank be used, they should be spaced a trifle apart, as they will swell when they become wet. If the planks are green they will shrink when wet, leaving thus cracks between them. The difficulty of the swelling or shrinking of the planking may be overcome by placing alternately well-seasoned and green planks.

The top of each post should be levelled in the down-stream direction, to let water run freely off the post. The holes in which the posts are set should be cut in a dovetailed fashion, the dovetail being on the up-stream side. The end of each post is cut to fit the dovetail, a shoulder 2 inches deep being made on that side. The hole cut in the rock must be larger, of course, than the foot of the post. In order to secure the post firmly in the hole, a long, wide key, about $2\frac{1}{2}$ inches thick, is placed on its lower side. It is important that this key should be well-fitted to the hole and that it be placed on the lower side of the post so that the pressure against the dam forces the key and post together. If the dovetail of the hole and the key are placed on the up-stream side of the post, the water-pressure will tend to force them apart.

To insure water-tightness and to prevent shocks from floating bodies it is advisable (but not absolutely necessary) to place a slope of earth and gravel against the up-stream side of the timber-dam. No apron is provided in this case, as the dam is supposed to be erected on a bed of hard rock. Should the rock be soft it must be protected by an apron.

Fig. 32 shows a simple kind of hollow frame dam. It can be used for any kind of a foundation, and requires much less timber than a dam made of logs. In Fig. 32

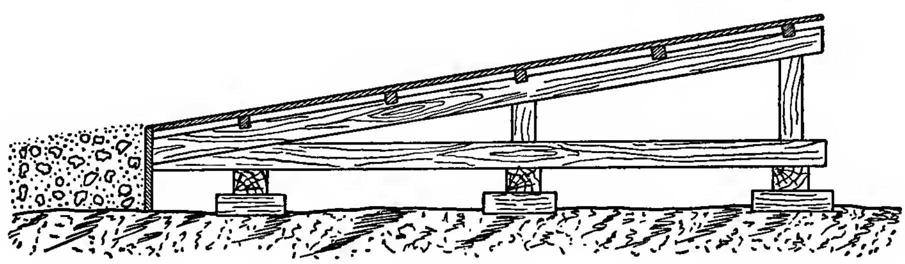


FIG. 32.

the dam is supposed to be built on a rock foundation. Short sills $10'' \times 10'' \times 4'$ are securely bedded parallel with the current and 8 feet apart, centre to centre, in both

directions. On these be -blocks the cross-sills of 12 by 12 inch timber are laid. Where joints occur on these sills 2-foot splices should be made, a block of wood being put under each joint. The cross-sills should be anchored to the rock by split-bolts $1\frac{1}{4}$ to $1\frac{1}{2}$ inches in diameter, which should pass through the cross-sills, bed-block, and about $3\frac{1}{2}$ feet into the rock. For the down-stream sill bolts should be placed 8 feet apart, but for the up-stream sill a bolt should be put in every 4 feet, a block being put under the cross-sill at every point where an anchor-bolt is placed.

The cross-sills support the bents, framed of $10'' \times 10''$ timber, which are placed 8 feet apart. Cross-ties of $4'' \times 7''$ timber are fastened about 4 feet apart to the caps of the bents. The dam is completed by spiking $1\frac{1}{2}$ to 2 inch planks to the cross-ties. It is not advisable to use planks of much more thickness, as they are more apt to rot on account of the wood being wet on one side and dry on the other. The remarks made on page 143 about the spacing of the planks apply, of course, also to this case.

Fig. 33 shows another manner of framing the bents of a timber-dam. When it is necessary to break the force of the water the down-stream face of the dam can be

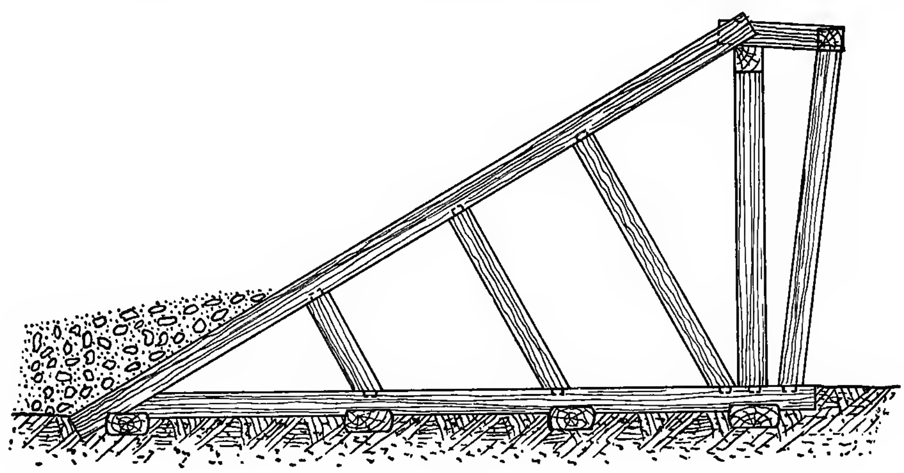


FIG. 33.

formed of a series of steps or an incline, as explained on page 137. A dam built at New Hartford, Conn , is a very good example of the latter style of construction. We shall describe it somewhat in detail to illustrate fully the construction of timber-dams.

The Dam at New Hartford, Conn., was built in 1847 across the Farmington River for the Greenwoods Company. At the place where the dam was constructed the river bottom consists of cobble-stones, gravel, and quicksand. The banks are composed of gravel and sand.

The dam has a length of 232 feet on its crest. It was originally built 21 feet high, according to the trapezoidal profile shown in Fig. 25, the width at the bottom being 68 feet. Both faces of the dam make an angle of 27° with a horizontal plane. The timbers of which the dam is built are 9 to 12 inches thick. The courses of timbers were laid alternately crosswise and lengthwise of the stream, the first course being laid across the stream. The timbers parallel with the current are 6 feet apart, those at right angles to the stream are 2 to 3 feet apart. Where the timbers cross each other they were fastened

with $\frac{3}{4}$-inch round spikes, 20 inches long. Both faces of the dam were originally covered with 3-inch oak and chestnut planks, placed close together and fastened with 7-inch cut spikes. All the spaces between the timbers were filled with stone. The crest of the dam was formed by a strong cap-log.

The apron extends 14 feet in front of the dam. It consists of timbers 12 inches thick placed close together. The mudsill supporting the down-stream end of the apron timbers is supported by piles driven about 15 feet into the river bottom. The apron was well tied to the dam by using, every 6 feet, long timbers extending 25 to 30 feet into the dam. The other apron timbers run only 2 or 3 feet into the dam. Sheet piles were driven at the up-stream toe of the dam. A slope of gravel, reaching within 4 or 5 feet of the cap-log was filled in on the up-stream face. Substantial masonry abutments were built on pile foundations on both sides of the dam.

During freshets 6 feet of water frequently passes over this dam and as much as 10 feet has been recorded. As the apron of the dam did not extend far enough, the water washed out a considerable quantity of gravel in front of it, and the proprietors were obliged to build cribs of logs, filled with large stones weighing 2 or 3 tons, to protect the dam against being undermined. These cribs were chained to the piles supporting the apron.

After the dam had been standing about twenty years the upper 10 feet had to be renewed, as the timbers had become rotten. This was supposed to have been caused by the hot vapor forming in summer inside of the dam, which faces south. The planking was therefore removed from the down-stream slope of the dam to allow the vapor to escape.

Dams across the Schuylkill River.*—On Plate 82 we show sections of a number of timber-dams which have been constructed across the Schuylkill River, to obtain slack-water navigation. No. 1 was built in 1819 at Plymouth. It was constructed without a coffer-dam on a bed of rock. The bottom timbers (12×16 inches) were placed 8 feet apart, parallel with the stream and secured to the rock bottom by two-inch oak treenails. The other courses of timbers were laid alternately crosswise and lengthwise of the stream, the timbers being securely fastened together with treenails, no iron bolts being used in the dam. The up-stream face of the dam was covered with timbers 10 inches thick placed close together. Until this sheathing was laid the water could pass freely between the timbers, as no stone filling was placed in the dam. The covering was done from both ends until only 60 feet of the dam was left uncovered for the water to pass through. The remaining sheathing was carefully cut and fitted and placed quickly in the dam by a large force of men before the river could rise so as to interfere with the work. A slope of clay and stone was placed against the up-stream face.

This dam stood for thirty-nine years, withstanding successfully floods that rose to a height of 11 feet above its crest. Although it was built upon a tolerably firm micaceous rock in nearly vertical strata, covered ordinarily by about 2 feet of water, the rock in front of the dam was worn out in thirty-nine years to an average depth of 3 feet (nearly an inch per year).† The depth of the water on the crest was usually 6 to 18 inches deep. The structure was replaced in 1858 by the dam shown in sketch No. 3.

* The facts stated about these dams and Plate 82 are taken from a paper on "Dam Building in Navigable and other Streams," by Edwin F. Smith, published in the Proceedings of the Engineers' Club of Philadelphia, for August, 1888.

† The Civil Engineers' Pocket Book, by John C. Trautwine, page 382.

The dam shown in sketch No. 2 was built in 1836. It was filled with stone and protected against leakage by sheet-piling. The framing in this dam was rather expensive, as the timbers had to be accurately fitted and joined. In 1846 the dam was raised for an enlarged navigation. It is still in an excellent state of preservation and has required but little repairs.

Sketches 3, 4, 5, and 6 show dams of more recent construction. They illustrate the type of dam now adopted for the Schuylkill Navigation. One of their characteristic features is that the up-stream face is made vertical or almost so. Experience has shown this type of dam to be cheaper, heavier, and stronger than the earlier kinds of dam built across the Schuylkill. It was urged against this type that the wide crest or comb would be damaged by debris or ice in floods, but long experience has proved that dams built according to this style are injured less than those having narrow crests.

Sketch No. 5 shows the Felix Dam which was built in 1855, 6 miles above Reading, Pa. It is 19 feet high and 27 feet wide at the base. It is similar in construction to No. 4. Although it has been subjected to heavy ice-floods, it is still in a remarkably good state of preservation.

Sketch No. 6 shows the Kernsville Dam, which was built on a gravel bottom in a gap of the Blue Mountains. It required a heavy apron to protect it from being undermined. In this case the apron was formed by extending the foundation-cribs of the main dam. This is not generally considered to be good practice on account of the injurious effect that may be produced on the main dam by the concussion of the overflowing water. In this particular case no harmful effect was apprehended, as the fall of the water was insignificant.

The Columbia Dam (Sketch No. 7) was built in 1875 across the Susquehanna River at Columbia, Pennsylvania. It is 6,847 feet long. Its average height is only $7\frac{1}{2}$ feet above low water. The base was made 30 feet wide to give the dam sufficient weight and strength to resist the violent floods to which it is exposed. The crest of the dam is 16 feet wide and level. The up-stream slope is vertical; the down-stream slope has a fall of 21 inches in $13\frac{1}{2}$ feet. The principal longitudinal timbers at the crest are of 12 by 13 inch white oak, in lengths of 40 to 50 feet. A covering of 5-inch white oak plank is placed over the crest and down-stream slope, and securely fastened with $\frac{3}{4}$-inch bolts $12\frac{1}{2}$ inches long. The planks in the down-stream slope are placed $\frac{1}{2}$ inch apart, to permit the water to keep the timbers wet.

On the up-stream face sheet-piling of 4-inch white pine planks, carefully jointed, was driven to the rock. This face of the dam is protected by plates of $\frac{7}{16}$-inch iron, reaching well over the crest timbers and down upon the sheeting.

The structure described replaced an older dam built with a narrow crest and a long down-stream slope, as it was supposed that the ice would pass freely over it. Experience proved this not to be the case. The Susquehanna River is noted for its great ice-freshets. In 1857, 4219 lineal feet of the dam was destroyed by ice; in 1865, 2500 feet; in 1873, 946 feet; and in 1875, 1085 feet. In the last mentioned year 2649 additional feet of the dam was damaged by the carrying away of the down-stream slope. On account of these experiences the dam built in 1875 (Sketch No. 7) was made with a wide crest like those adopted on the Schuylkill, and the results have proved to be very satisfactory.

This dam has withstood some of the severest ice-floods ever known on the Susquehanna. In thirteen years the level crest suffered very little, but the 5-inch oak planking on the down-stream slope, while almost uninjured at its junction with the crest, was worn down to 1 or 2 inches thickness at the point of the overfall, leaving the iron bolts by which it was fastened projecting 3 to 4 inches and bent over down-stream. If the dam had to be rebuilt, the down-stream slope would probably be abandoned, a level deck being adopted for the whole width of the dam.

In building dams in rivers subject to destructive ice-floods, a timber dam should be made as heavy as possible. This object will be best accomplished for low dams by adopting a square cross-section and placing a slope of stone and gravel to help the ice over the dam. The upper part of this slope should be protected by a paving of stone. It is not advisable to use much timber in such a dam, as it reduces the weight of the structure. In the winter of 1887–88 two short sections of the Columbia Dam, 50 to 60 feet in length, raised 12 to 18 inches. This was ascribed to the preponderance of white-pine timber used in the dam at these points. When these sections raised the dam was submerged in 10 feet of backwater caused by an ice-pack some miles below.

The Holyoke Dam* (Plate 83) was built in 1849 across the Connecticut River by the Hadley Falls Company (now the Holyoke Water-Power Co.). Before this structure was begun a temporary dam was built a little further up-stream to serve as a protection during the construction of the permanent dam and to furnish water-power in the meantime. The temporary dam was built somewhat like the permanent dam, constructed subsequently, but was given less strength. The gates of the temporary dam were closed on November 16, 1848. When the water reached within 2 or 3 feet of the top, the whole dam, except 75 feet on one end and 150 feet on the other, was rolled over and floated down-stream on the crest of a wave about 8 feet high. The loss to the company on account of this failure is stated to have been $40,000 to $50,000.

The permanent dam shown in Fig. 1, Plate 83, was begun the following year and finished in the summer. This dam is still standing, but will soon be replaced by a masonry dam below it. It is 1017 feet long and has a maximum height of about 30 feet. The down-stream face of the dam was originally made vertical, but in 1870 a sloping apron was built in front of the dam, as shown in Plate 83.

The dam was founded on a ledge of red slate and sandstone, which dips down-stream about 30° from a horizontal plane. The whole dam was built of heavy timbers, nothing less than 12 by 12 inches being used. The bottom timbers (15 by 15 inches in section) were placed parallel with the current and were bolted to the bed-rock with iron bolts 1¼ inches in diameter, about 3000 of these bolts being used in the dam. The bottom timbers and those directly over them were placed 6 feet apart and divided the dam into 170 sections. The up-stream slope, which makes an angle of 20° 45′ with a horizontal plane, was covered with three courses of 6-inch timber. This planking was strongly fastened together with spikes and treenails. The rolling top or combing was covered across the whole length of the dam with sheets of boiler-iron. Four million feet, board measure, of wood was placed in the dam.

As the dam was built up the pockets between the timbers were filled with stone to a

* Paper by Clemens Herschel, Trans. Am. Soc. C. E., for 1886, and Engineering News of May 13, 1897.

height of 10 feet. Above this the dam was originally left hollow. The foot of the dam was protected by concrete. A bank of gravel was filled in against the up-stream face of the dam, beginning 70 feet above the dam and covering over 30 feet of the slope. Strong abutments of masonry were built on both sides of the timber dam. The total cost of the dam amounted to $150,000.

During the construction of the dam the river-water was passed through 46 gates, each having an opening of 16 by 18 feet. These gates were closed for the first time on October 22, 1849, the water being thus forced to pass over the dam. The work stood this test very successfully. The leakage through the dam was very trifling, not more than was thought necessary to keep the timbers from decay.

In November, 1849, 6 feet of water passed over the top of the dam. It caused the windows in Springfield, 8 miles away, to rattle, as no provision had been made to allow the air to pass freely from abutment to abutment under the sheet of water. In April, 1862, 12½ feet of water passed over the dam, which is the maximum height the water has reached.

The water, ice, logs, etc., passing over it rapidly wore away the rock in front of it. By 1868 the ledge had been eroded to a depth of 20 to 25 feet, and the dam had become undermined in some places. Besides the wearing out of the rock, the front timbers had become injured by logs and ice which, after passing over the dam, were forced against the front face by the eddies caused by the falling water. In some cases logs having become wedged among the front timbers and being struck by the falling water forced the timbers apart, acting like large levers. In order to protect the dam against such injuries and to reduce the fall of the water, a large inclined apron of cribwork was built in front of the dam during the years 1868, 1869, and 1870 (Plate 83). This crib, which exceeds the original dam in volume, was built of round logs laid so as to form pockets 6 by 6 feet, which were filled with stone to the top before the covering, consisting of 6-inch planks of hard wood, was put on. The cost of the apron is given variously as $263,000 to $350,000—about double the cost of the original dam.

The construction of the apron merely transferred the erosive action of the water further down-stream. The slope of the apron being nearly parallel with the dip of the rock, the circumstances for washing out the ledge were very favorable. By 1886 the rock in front of the apron had been eroded in places to a depth of 20 to 25 feet.

While the apron was being constructed a considerable amount of stone was also filled into the old dam. This work was done carelessly, stones weighing 4 to 5 tons and even whole scow-loads of stone being occasionally dropped on the up-stream slope of the dam. The leakage through the dam, which in after years entailed much expense, was probably partly due to the injuries thus sustained.

From 1849 to 1879 only trifling repairs were required on the dam with the exception of the construction of the apron. In the latter year a break occurred in the plank covering of the up-stream slope. Many similar breaks taking place in the next few years, the whole up-stream slope was replanked in 1885. At the same time sheet-piling was driven longitudinally through the dam, about three bents back from the face, and gravel dumped and puddled on both sides of the sheet-piling. The cause of the breaks was the rotting of the planking (which, as already stated, had been injured by stones being

dropped on it) and also of some of the timbers. For a full account of how the dam was repaired we must refer the reader to Mr. Clemens Herschel's Paper (Trans. Am. Soc. C. E., for 1886).

As the repairs appeared to have no permanent effect in stopping the leakage, it was finally decided to build a masonry dam 112 feet at one end and 132 feet at the other down-stream from the old timber structure. Surveys for the new dam were made in 1891. The construction was begun in 1895. It is expected that the dam will be completed during the season of 1899.

Figs. 2 and 3, Plate 83, show the profile adopted for the masonry dam. The upper part of the down-stream face is the parabola, which a sheet of water 4 feet in depth over the crest would describe in falling freely. The parabola continues to the point of reversing below which a cycloid (the curve of "quickest descent") is adopted for the face. At the extreme toe the face is turned somewhat upwards to break the force of the water and to prevent it from cutting the ledge beyond. The back slope forms a series of steps, 5 feet high, equivalent to a batter of 1 foot in 5 feet. The length of the rollway of the new dam will be 1020 feet. The plans for the masonry dam* were prepared by Mr. E. S. Waters, Chief Engineer of the Holyoke Water Power Company.

In conclusion, it may be of interest to mention some of the lessons taught by the old timber dam and summarized by Mr. Herschel in his "Paper":

1st. A wooden dam should not be left hollow, as the foul air on the inside will eventually rot the timbers. A stone filling will not prevent this decay, but a tight filling of gravel will protect the timbers against rotting.

2d. A masonry shelf on a masonry abutment should not take the place of the last frame of a dam. The dam will probably settle, but the masonry will not, and thus a distortion will be produced in the framing of the dam.

3d. The down-stream face of the dam should never be vertical unless the height be very insignificant.

4th. An apron should be provided and given a proper form to prevent the water from washing out the river-bed in front of it.

A long, steep slope of timber on the up-stream side of a dam is very objectionable. Mr. Edward F. Smith has pointed out that if the plan of the original Holyoke Dam had been turned around so that the up-stream face would have been downstream, and if a broad comb 10 to 15 feet wide had been added at the up-stream (vertical) face of the dam, adding about 30 per cent to the mass of timber and stone, the cost of the expensive crib-apron would have been saved.†

A slope of gravel on the upstream side instead of the long timber one would have made the dam tighter and have avoided all the expensive repairs which became afterwards necessary.

The simple triangular profile shown in Fig. 24, page 137, is often adopted for low dams, but the experience with the Holyoke Dam proves clearly that such a profile should never be used for a high fall of water.

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* For a full account of the manner in which the masonry dam is being built, see Engineering News for May 13th, 1897.

† Proceedings of Engineers' Club of Philadelphia, for August, 1888.

The Canyon Ferry Dam* was built across the Missouri River near Helena, Montana, for the Helena Water and Electric Power Company. All the plans for the work were prepared by Mr. J. T. Fanning, the Consulting Engineer of the Company.

Fig. 34 shows a section of the dam which consists of timber cribs filled with stone. It is 485 feet long and 29 feet high. The timbers are fastened together with iron drift-bolts 20 to 30 inches long. The down-stream face of the dam forms three steps to break the force of the water and prevent it from scouring out the river-bed. The steps are covered with 20 inches of timber (two courses 10″ × 12″ timbers laid on the 12-inch side). The risers are covered with two courses of 3-inch plank, lap-jointed. The back of the dam is covered in a

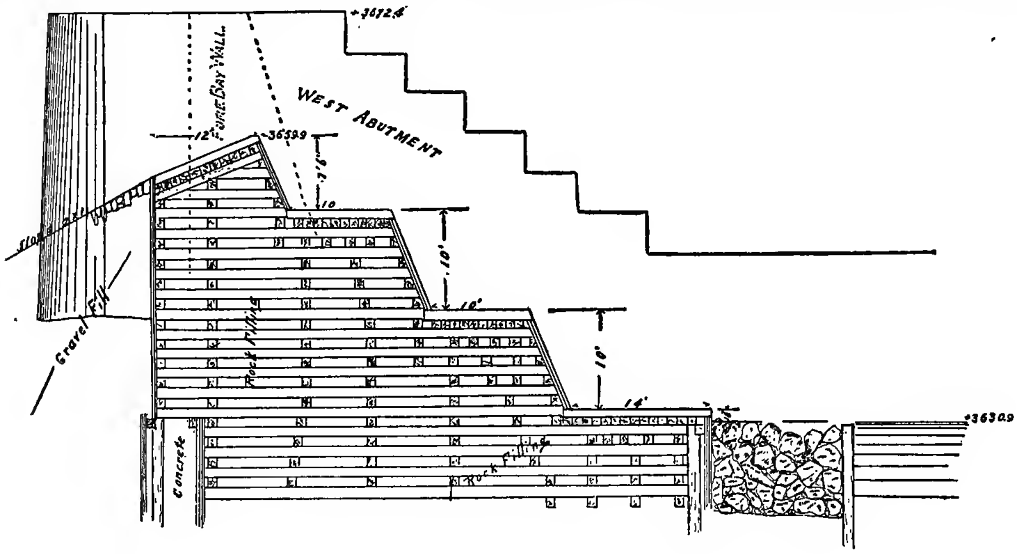

Fig. 34.—Canyon Ferry Dam.

similar manner with 2-inch plank. An earth slope, riprapped at the top, was placed against the back of the dam, and below the dam large rocks were filled in to the surface of the river for a distance of 25 feet, being held in place by a row of round piles.

The timber-dam was founded on a bed of gravel and granite sand, which is almost impervious to water. Both above and below the crib-dam a row of triple-lap sheet-piling made of 3 by 12 inch plank, stiffly bolted together, was driven to a depth of about 12 feet below the river-bed.

Masonry abutments were built on both sides of the crib-dam to a height of 12½ feet above its crest. On the east bank an earth dam 285 feet long, having a masonry core-wall and slopes of 2 to 1 and 1½ to 1 respectively on the up-stream and down-stream sides, was built to the hillside, the top of the dam being at the level of the top of the abutments.

* Paper read by Mr. Theron M. Ripley before the Montana Society of Engineers, January 8th, 1898; see Journal of the Association of Engineering Societies, May, 1898.

PART III.

MOVABLE DAMS.

CHAPTER I.

FRAME-DAMS.

Canalization of Rivers.—On many rivers navigation becomes impossible at shoals during periods of low water. In early times boats had to be kept at such places until rain-storms raised the river sufficiently to carry them over the shallow points. The first improvement attempted to remedy this trouble consisted in building dams (weirs) across the river where needed to increase the depth of water for "slack-water navigation." Each of these dams had usually one or more openings, which could be temporarily closed by "stanches" consisting of spars, planks or gates, bearing at the bottom against a sill and at the top against movable wooden beams. By removing these beams suddenly, and thus releasing the stanches, an artificial flood was produced which carried any boats that might be above the dam through the openings and over the shoals below. This process was called "flashing," or "flushing." It was in use on several rivers in France until the middle of this century and also in England on the Thames and Severn.* Instead of vertical planks, etc., for stanches, horizontal beams (poutrelles), placed one on top of another, were sometimes used in France to close openings of 15 to 18 feet in a dam. By a suitable arrangement, these beams could be suddenly released for flashing.

In order to make it possible to take a boat up or down stream without removing the stanches, a lock was often built at these dams. By constructing at suitable points dams with locks or stanches, a river was practically converted into a canal,† with the difference, however, that it still remained subject to floods, for which provision had to be made.

Needle-dams.‡—About the end of the eighteenth century the French Government commenced to improve internal navigation by constructing substantial "navigable

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* Minutes of Proceedings Inst. C. E., Vol. IV., p. 111.

† The river Lot, in France, was the first river to be canalized in this manner. Dams were built across this river in the thirteenth century and locks were introduced in the fifteenth century.

‡ The authorities on movable dams, which the writer has consulted, are given on page 242. For the historical notes on works of this character in France, he is indebted to Lagrené's "Cours de Navigation Intérieure" and to memoirs on Movable Dams that have appeared since 1839 in the "Annales des Ponts et Chaussées."

passes," 26 feet wide, in some of the dams built across rivers, especially on the Yonne, a branch of the Seine. These passes were built with side-walls and aprons of masonry. They were closed by small wooden spars called needles, which bore on the bottom against a masonry sill and at the top against wooden beams, pivoted on iron pins placed in the side-walls. Later on, the width of the passes on the river Yonne was increased to 40 feet, a cable which could be slacked or stretched as required being substituted for the pivoted beams. As a pass only 40 feet wide was found to involve considerable inconvenience and danger to navigation, M. Poirée, who had charge of the improvements on the river Yonne, increased the width of the pass at Basseville, which was constructed in 1834, to 72 feet. For such a width a cable could no longer be used for supporting the upper ends of the needles. M. Poirée substituted for the cable a series of short iron bars, which were fastened to the top of iron frames (trestle-bents), placed at short intervals across the pass, from one side-wall to the other. In the Basseville dam these frames were originally two metres (6.56 feet) apart, but this distance was afterwards reduced to one-half. In needle-dams erected subsequently the distance between the frames varies from 3 to 4 feet.

One of the chief features of M. Poirée's invention was the manner in which he removed the frames when the pass was to be opened. This was accomplished by removing the needles by hand, unhooking the bars connecting the frames, and turning the latter down on journals placed in their lower bases until they rested in a recess in the masonry apron, presenting thus no obstacle above the sill of the floor. It was objected at first that the frames would be silted up while lying thus on the apron, and that it would be very troublesome to raise them again, but experience on the Yonne and similar rivers has proved that no difficulty has been experienced in this respect, as the bents are only turned down during periods of high water, when but little silt is deposited.

In Fig. 35* we show one of the earlier Poirée dams. The frames are made of bar iron about $1\frac{1}{2}$ inches thick. Each frame has a trapezoidal form, the top and bottom pieces being horizontal, the up-stream post vertical and the down-stream post slightly inclined. A diagonal brace serves to stiffen the frame. The ends of the lower base form journals which fit into cast-iron boxes fixed in the floor. The frames are 6.23 feet high, 2.56 feet wide on top, and 4.92 feet wide at the base. They are placed a metre (3.28 feet) apart, and weigh each 242 pounds. Wooden planks placed on top of the frames form a bridge which enables the dam-tender to replace or remove the needles, etc. Two men are able to raise or lower a frame by means of iron chains joining their tops.

When erected the frames are fastened together on top by connecting bars, both on the up-stream and down-stream sides. The up-stream bars are made stronger than those on the down-stream side, as they have to support the upper ends of the needles. The bars are made in various ways. In the dam shown in Fig. 35 the bars are pivoted at one end and provided at the other with a hook which can be attached to a pin on the cap of the adjoining trestle. In some of the earlier dams

* Figs. 35, 41, and 63 are taken from the "Paper by B. F. Thomas on Movable Dams," Trans. Amer. Soc. C. E. for 1898.

the connecting bars had pinholes at both ends, and were placed over pins attached to the caps.

The construction of the first Poirée needle-dam was soon followed by others, the details being perfected and frames of greater height being used to secure a greater depth

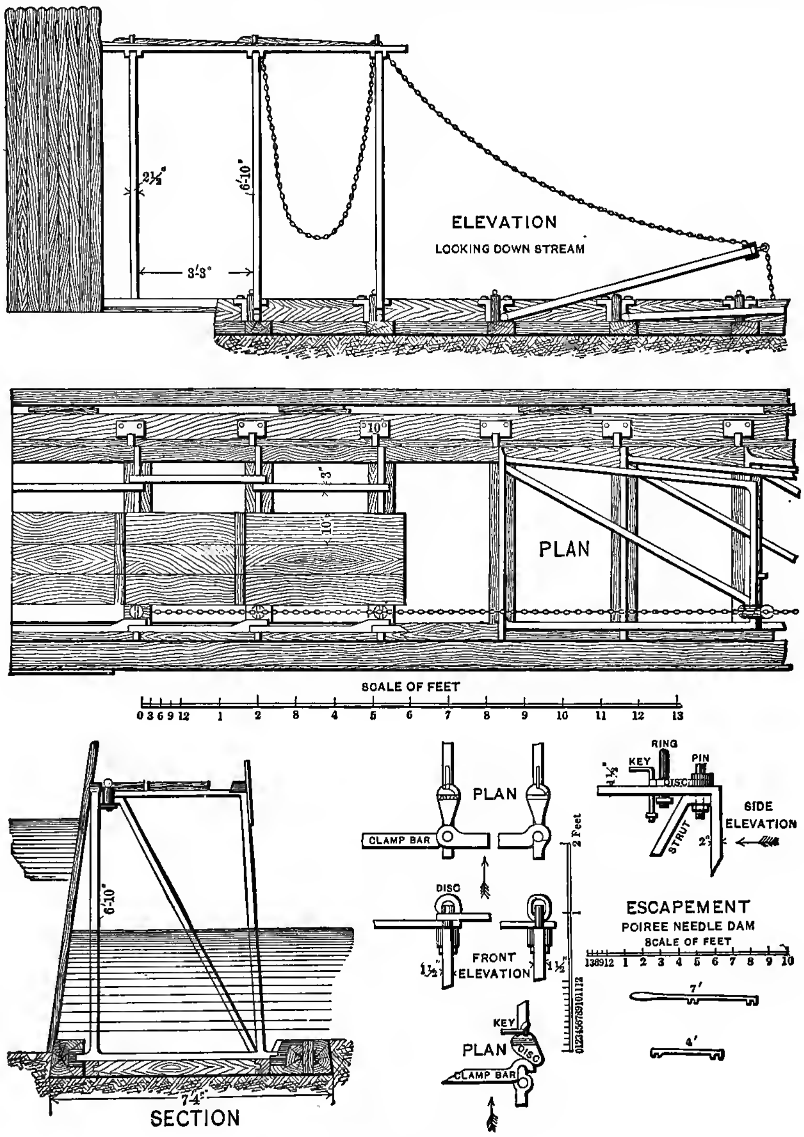

FIG. 35.—POIRÉE DAM.

of water for navigation. The second needle-dam was built at Decise, on the Loire, in 1836. A similar work, described below, was constructed in 1838 at Épineau, on the Yonne. In 1840–45 needle-dams were built to replace the fixed weirs on the Saône, which caused great damage during floods. Needle-dams have been erected on the following streams in France: The Seine, Marne, Oise, Cher, and Allier; and also on the Belgian Meuse and on the Main in Germany. A dam of this kind constructed in the United States is described on page 159.

The Needle-dam at Épineau, on the Yonne, built in 1838, is one of the earliest works of this kind. A description of the dam, written by M. Chanoine, the engineer

in charge of the construction, is given in the "Annales des Ponts et Chaussées" for 1839, First Series, p. 238. The dam had a length of 230 feet and a height of $6\frac{1}{2}$ feet above the sill. The frames, which were placed a metre apart, were made of bar iron $1\frac{1}{2}$ inches square. Each frame was 7 feet high, 4 feet 7 inches wide at the base and 4 feet 3 inches wide on top. The wooden needles were $2\frac{3}{4}$ by $1\frac{1}{2}$ inches in section and 8 feet long, each weighing about 13 pounds.

In several similar dams that were constructed subsequently on the Yonne, the frames were made 7 feet $4\frac{1}{2}$ inches high and placed 3 feet 7 inches apart.

The Needles used in Europe have generally been made of red pine. In the first dams they were only about $1\frac{3}{4}$ inches square and 8.2 feet high, each needle weighing, when wet, about $4\frac{1}{2}$ pounds. As higher dams were constructed, heavier needles had to be used. The largest needles in France are $4\frac{3}{4}$ inches square and 16 feet 5 inches long, each weighing about 100 pounds. A needle of this size and weight can still be placed by hand, but heavier needles have to be handled by machinery. Needles 8 inches square have been experimented with in France, but abandoned as too heavy. In an American dam (page 160) needles $4\frac{1}{2}'' \times 12'' \times 14'\ 3''$, weighing each 263 pounds when wet, have been used, but they are placed by means of machinery. The needles have a square or rectangular cross-section. For high pools the thickness of the needle is reduced according to the strain to which it is subjected, the width remaining, however, the same. Needles having a hexagonal or semi-hexagonal section have been experimented with but have not yet been introduced.

The top of the needle is formed into a handle. It is generally provided with an iron ring and sometimes with a hook serving to attach the needle to the supporting-bar. When the needles are to be released mechanically by turning the supporting-bar, those between two adjoining frames are usually fastened to the same rope, which passes through the iron rings or through eyes in the needles. This rope is attached to a hawser, one end of which is fastened on shore. By this arrangement the needle can easily be recovered.

As the height of Poirée dams became greater two difficulties were encountered:

1st. The needles were often broken in handling;

2d. The leakage between the needles increased considerably.

To avoid breakage heavier needles, proportioned according to the pressure they were to sustain, were introduced. In some dams a wooden bar has been placed on the up-stream side of the frames to relieve the needles of some of the pressure they have to bear. The bar is suspended by chains and bears against the needles at about

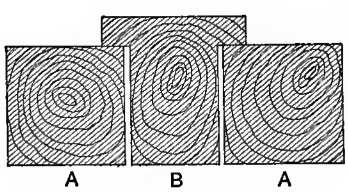

FIG. 36.

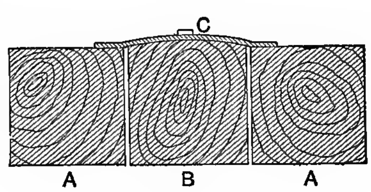

FIG. 37.

one-third the height of the part under pressure. Hollow needles have been proposed, as giving greater strength for the same weight than solid spars, and it has also been

suggested for high dams to use two sets of needles, one for the lower and one for the upper part of the dam.

The leakage can be diminished by placing straw, etc., in front of the dam. Alternate needles of a "T" form (Fig. 36), or with india-rubber facing ("C" in Fig. 37), have been proposed. None of these improvements suggested to avoid leakage and breakage have yet been practically introduced.

Frames.—In the first needle-dams the frames were only about 7 feet high. As such dams were built later on to retain greater depths of water, the height of the frames had to be increased. In the Martot Dam, on the Seine, the frames are 11 feet high. The frames of the dam at Louisa, Kentucky (page 159) are 15.17 feet high.

The simple construction of the early frames had to be modified as their height was increased. "T" or "U" iron, etc., were used in the frames as giving

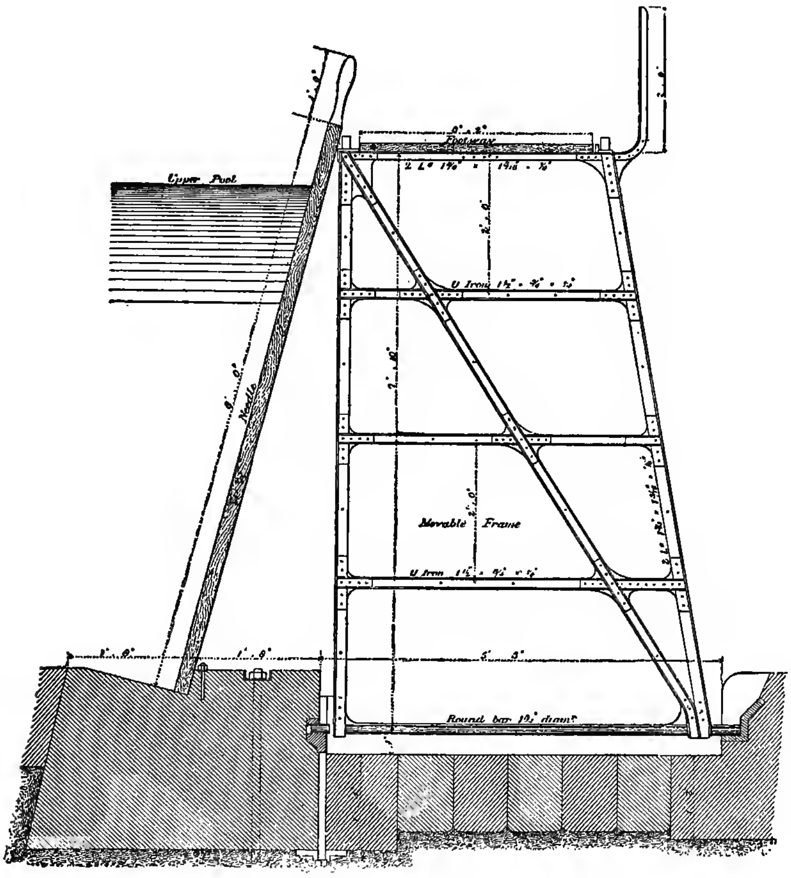

FIG. 38.—POIRÉE NEEDLE-DAM.

greater strength for the same weight than bar iron. More bracing was also required. Fig. 38* shows a modern frame. In some dams the frames are surmounted by light framework in order to raise the foot-bridge so as to be above all danger of

*Figs. 38, 51 to 53, and 56 to 60 are taken from "Fixed and Movable Weirs," by L. F. Vernon-Harcourt, in Minutes of Proc. Inst. C. E. for 1880. Figs 47 to 49 are taken from a paper on the River Seine by the same author, in Minutes of Proc. Inst. C. E. for 1886.

submersion. Standards for a rope-railing were also attached to the frames on the down-stream side in order to reduce the danger to which the dam-tenders were exposed in handling the needles at night or in stormy weather.

The chains that were attached to the caps of the frames of the early dams have been omitted in some more modern constructions in France, as the frames can be readily raised by means of boat-hooks.

Improvements were soon introduced in the details of needle-dams. One of the most important was the substitution of an iron foot-bridge for the wooden planks (Fig. 39, page 158), an invention made by M. d'Haranguier de Quincerot. The iron bridge was arranged in such a manner as to fasten the frames together when up and to fall with them when lowered, partially covering them when down. The frames of the dams on the Cher are arranged in this manner.

Another improvement was the introduction of a releasing contrivance, which allowed the supporting-bar between any two adjoining frames to swing loose, setting thus the needles free, which were attached to a rope and could be easily recovered. This contrivance became more important as the height of the Poirée dams was increased. MM. Poirée and Chanoine invented such contrivances for dams in France whereby a length of dam of 130 feet could be opened in fifteen minutes, instead of requiring an hour by the primitive method of removing the needles by hand. A very simple release device, invented by M. Kummer and used in dams on the Belgian Meuse, is described on page 158.

In the early Poirée dams no special provision was made to carry off flood-waters except by the overflow formed by the fixed part of the dam. As the foot-bridge of the needle-dam had to be kept low (about 12 to 18 inches above the water), it was exposed to the risk of being submerged, which would make it impossible to lower the dam. To avoid this danger the upper part of the overflow-weir was made movable by placing on its crest some kind of shutter, such as the Chanoine wicket (described on page 174), which could be readily removed.

Method of Working.—Two attendants were able to perform by hand all the work of raising or lowering the first Poirée dams. Supposing the frames to be down, the attendants erected the dam in the following manner: They first raised the frame nearest the abutment (which was the last one to be lowered) by means of the chains fastened to it, or with boat-hooks, attaching it temporarily by a hook or clamp to a ring fixed in the abutment. The planks of the foot-bridge were next laid from the abutment to the erected frame, which was then firmly attached to the abutment by the up-stream and down-stream connecting-bars. The other frames were then raised and attached in succession in a similar manner. A handle with projections (Fig. 35) served to hold a frame temporarily in place until the planks were laid and the connecting-bars were attached.

In placing the needles every other one would first be put into position so as to dam the water gradually, the intermediate needles being finally placed. The needles, even including those weighing up to 100 pounds, were placed by hand in the dam by shoving them into the water so as to allow the current to bring their lower ends against the sill. In the large dams on the river Marne (France) each needle is pro-

vided with a handle and an iron hook. In placing a needle, which in these dams weighs about 103 pounds, it is held horizontally until the hook has been attached to the supporting-bar, and then the point of the needle is lowered into the current, which carries it against the sill. The distance from the hook to the point of the needle is made about half an inch longer than the distance from the supporting-bar to the sill. This causes the foot of the needle to scrape along the floor before striking the sill and thus avoids all shock. When ligther needles are used they may be pushed almost vertically into the water so as to bear against the sill.

The attendants soon acquire considerable skill in performing their work. They are nevertheless exposed occasionally to danger in lowering a dam at night and in stormy weather. The necessity of doing so was, however, later on almost eliminated by providing ample overflow-weirs having movable parts, which were more easily handled than needles, and by organizing a system of signals by telegraph, telephone, etc., along the river, giving ample notices of freshets likely to occur.

As the weight of the frames and needles increased, some power had to be supplied for handling them. This has usually been done by means of a windlass placed on a little truck moving over the trestles on rails. This truck serves also for transporting the needles. In some cases the dam-tenders have worked from a boat placed on the down-stream side of the dam, but experience proved that they were exposed to more danger, especially in lowering the dam, in a boat than when working from a bridge.

Needle-dams in Belgium (Fig. 39). In 1875–78 twenty-seven needle-dams were constructed on the Belgian Meuse. They contain all the improvements made up to that date. Each of these works consists of: A lock, having an available length of 328 feet and a clear width of 39.33 feet; a navigable pass 150 feet long, which can be closed by a needle-dam; and an overflow-weir 179 feet long, on top of which Chanoine wickets (page 174) are placed. The sill of the weir is laid at low-water; the sill of the pass is placed 2 feet lower. When the dam is up the pool formed is 10.17 feet above the pass-sill.

The frames of the pass are placed 3.93 feet apart, centre to centre. They are 8.36 feet wide at the base, 4.76 feet wide on top, and 11.48 feet high from the floor to the under side of the collar of the bar supporting the needles.

The frame is made of wrought-iron bars. It is stiffened by a diagonal brace consisting of two pieces. A horizontal tie passes between the two bars of the brace, as shown in Fig. 39.

The top of the frame, when up, reaches the normal level of the water in the pool. The foot-bridge is kept about 18 inches above the water by placing on top of the frame two short iron posts, each 19.7 inches long, one at the up-stream and the other at the down-stream end of the cap. The former consists of a piece of tube and is part of the arrangement for releasing the needles, as explained hereafter. The latter is made of a solid piece of iron. The axle from which the iron floor is hung connects these short posts. The total weight of a frame, including the floor, escape-bars, etc., is 1108 pounds. The up-stream journal-box for the axle on which the frame turns is let into the sill and held by screws and bands. The down-stream journal-box is bolted to the stone. These boxes weigh respectively 70 and 200 pounds.

The frames are held rigidly together by a sheet-iron floor 3.64 feet wide. One end of each section of floor is permanently attached to a frame by the axle on which it revolves; the other end terminates in claws which grasp the cap of the next frame. The floor is connected at the abutment, pier, and lock-wall to iron bars like the caps of the frames, which are fastened to the masonry.

The needles are made of red Riga fir. They are 12.3 feet long and $3\frac{7}{8}$ inches wide. At the point of maximum pressure and for 10 inches each way the needles are $4\frac{3}{4}$ inches thick. They are $3\frac{7}{8}$ inches thick at the bottom and $3\frac{1}{2}$ inches at the top. Each needle is finished at the top so as to form a handle 9 inches long, ending in a

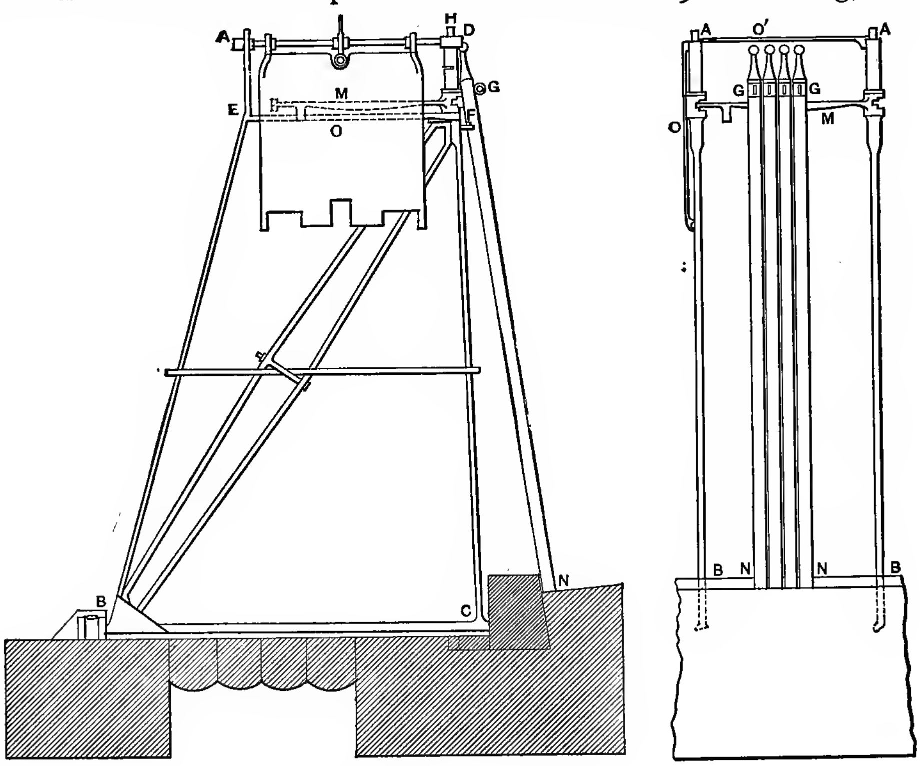

FIG. 39.—BELGIAN NEEDLE-DAM.

ball. It is provided with an iron ring, through which a rope passes that connects the eleven needles of each bar. One end of this rope is tied to the down-stream leg of the trestle, the other end is knotted.

The needles, which weigh 55 pounds apiece, are placed by hand by the dam-tender. When it is desired to release the eleven needles between any two frames, the rope connecting the needles is fastened to a hawser tied at one end to the pier or shore. The escapement is then turned and the needles are carried by the current below the dam.

The escape device used in these dams was invented in 1845 by M. Kummer, the Chief Engineer of the Meuse Improvements in the Province of Liège. It is constructed in the following manner: The bar *M* (Fig. 39), supporting the tops of the needles between any two frames, is connected at one end by means of a collar to the hollow tube *D* bearing the up-stream end of the floor-axle in such a manner that it can turn horizontally when released. At the other end it rests (when locked), against a

circular post called a jack-post, placed inside the tube *D* of the next frame. A semi-circular notch is cut in the jack-post at the elevation of the support-bar. Similar notches are cut in the tube *D*, in which the jack-post is placed, and in the rear end of the collar of each support-bar, *M*. The head *H* of the jack posts, which projects out of the tube *D*, is made square to enable the dam-tender to turn it by means of a wrench or key. When the jack-post is turned so that its notch corresponds to that of the tube the support-bar of the needles becomes free and swings back horizontally, releasing the needles.

Needles are used only for the pass. The movable dam placed on top of the weir consists of 39 Chanoine wickets (page 174) each 7′ 4″ high by 4′ 3″ wide. A 4-inch space is left between two adjoining wickets. It may be closed by a board during low-water. The wickets are maneuvered from a frame foot-bridge placed on the up-stream side of the weir.

The Dam across the Big Sandy River at Louisa, Ky.,* built in 1891 to 1897, is the first needle-dam constructed in the United States. It differs in several respects from similar works in Europe. It sustains a greater head of water, the needles are much wider and heavier, the trestle-frames are much lighter, and the methods of operating the dam are new. According to the original plans, needles were to be used for the pass and wickets for the overflow-weir, but it was finally decided to use needles both for the pass and weir. We believe that this is the first dam in which this arrangement has been adopted.

The works consist of: A lock 52 feet wide by 255 feet long, located on the right bank of the river; a navigable pass, next to the lock, 130 feet long, and an overflow weir 140 feet long, separated from the pass by a pier 12 feet wide and terminating at an abutment 17½ feet wide, on the left bank. The total length of the masonry foundation, including the lock, is about 400 feet.

The sill of the pass is about one foot below the low-water mark of recent years. The sill of the weir is placed 6 feet above that of the pass. The normal height of the pool is 13 feet above the sill of the pass.

The frames are placed four feet apart between centres. Those of the pass are 15′ 2″ high and 9′ 10½″ wide at the base. The weir frames are 9′ 8″ high and 6′ 5″ wide at the base. The weights of the pass and weir frames are respectively 1140 and 920 pounds.

The frames are made of 4-inch steel channels, the up-stream parts being single, while those on the down-stream side are made of two pieces, set apart and trussed as shown in Fig. 4, Plate K. The posts of each frame are connected by two horizontal braces made of angle-iron. A suitable frame for carrying the floor is riveted to the outside of the main trestle-head.

The bar which connects two adjoining frames with each other when standing and supports the upper end of the needles is hinged vertically at one end at the pool level so as to swing horizontally. On the other end it is formed into a hook, on its up-stream side, which engages with a lip or projection on the next frame. A crank-

* Paper on Movable Dams by B. F. Thomas, Trans. Am. Soc. C. E., June, 1898. Report of the Chief of Engineers, United States Army, 1897.

shaped rod, called a jack-post, serves for holding in place or releasing the hook end of the bar. When the dam is up this post is kept by a latch from turning. When the needles are to be released the latch is raised and the jack-post is turned by a wrench so as to allow the hook end of the bar to pass through the space formed by bending the post.

The frames are connected on top by a sheet-iron floor, which is hinged to and falls with them. They are also connected by the maneuvering chain.

The frames are raised or lowered by means of two chain-crabs—one for the pass and the other for the weir. The former is located on the lock-wall; the latter is situated on the pier. As the frames of the pass and weir are raised or lowered by their respective crabs in the same manner, we shall only describe the method of raising those of the pass. A chain which can be wound or unwound by the crab passes over all the frames and is attached to the one furthest from the crab. This chain passes, at each frame, over a combined chain and ratchet-wheel, which is attached to the head of the frame and turns on a horizontal axis. When the pawl is out of the ratchet the chain-wheel simply turns as the chain from the from the crab is moved, without producing any effect on the frame. When the pawl is dropped into the ratchet the chain-wheel is locked, and consequently any motion of the chain from the crab will revolve the frame on its journals. The chain passes at the crab over a sprocket-wheel and drops through a hole into a recess provided for it in the masonry.

The Needles are made of white pine. They are 12 inches wide. Those for the pass are 14′3″ long, $8\frac{1}{2}''$ thick at the bottom and $4\frac{1}{2}''$ on top, each needle weighing when wet about 263 pounds. The needles for the weir are 8′3″ long, $3\frac{1}{2}''$ thick at the bottom and $2\frac{1}{2}''$ at the top, each weighing about 80 pounds. All of the needles are banded at the top and bottom and are provided with iron handles at the top for convenience in handling. They have also suitable attachments for connecting-chains, for placing or removing them. Shallow grooves are cut in the sides of the pass-needles for strips of rubber, which may be placed in these grooves to prevent leakage. Thus far this has not been found necessary, the dam being remarkably tight.

The needles are placed by means of a boat on which those for the pass are stored when not in use.

Method of Working.—The following operations are required in working the dam:

1st. Raising or lowering the frames;

2d. Placing or removing the needles.

1st. *Raising or Lowering the Frames.*—Two men turning the crab and a third man to connect the frames, etc., can perform this work. When the frames are down the iron floor locks the pawls in the ratchet-wheels. Consequently, as the men at the crab wind in the chain the frame nearest the crab starts first to rise and others follow in turn. When the first frame is nearly vertical the attendant in charge of this part of the work raises the iron floor a few inches. The effect of this motion is to turn the pawl out of the ratchet, disconnecting thus the frame from the motion of the chain, which now simply revolves the chain-wheel. The attendant connects

PLATE K.

Fig. 1.

Fig. 2.

Fig. 3.

Fig. 4.

NEEDLE-DAM ON THE BIG SANDY RIVER AT LOUISA, KENTUCKY.

now this frame with the masonry by means of its floor and then places the connecting-bars. By the time he has performed this work the second frame has come within reach. This is stopped and connected to the first frame in the manner just described. The remaining frames are handled in a like manner. The iron foot-bridge is prolonged to the pier or abutment, as the case may be, by a rolling foot-bridge.

To lower the frames the work described is done in a reversed order. The attendant unhooks the rolling bridge and shoves it back into a recess provided for it in the masonry. He then disconnects the two frames furthest from the crab and unhooks the iron floor, which falls on the crab-chain, locking the pawl in the ratchet. The attendant pushes the last frame away from the crab while the chain is being unwound. When about 4 feet of chain have been unwound, the connections between the next two frames are removed and the pawl of the frame next to the one already being lowered is locked in the ratchet-wheel by dropping the iron floor on the crab-chain in the manner just described. This frame is pulled down by the weight of the one already descending, and so on. Several frames are usually raised or lowered at the same time, Plate K, Fig. 1. The men at the crabs need not stop in the winding or unwinding.

Should the chain break, as has happened in the first working of the dam, the frames can be handled from the needle-boat.

2d. *Placing or Removing the Needles.*—This has been done in two ways: 1st, directly from the boat; 2d, from the water by means of a derrick on the boat. The first method was the one originally contemplated, but the second has been found by experience to involve less work.

When the first method is adopted the needles are handled by means of two trolleys travelling on suspended tracks, one on each side of the boat. In placing the needles every fourth one is first put in position in the dam. The intermediate needles are first placed on shelves, temporarily attached to the trestles just above the water. When all the remaining needles are placed in this manner, each shelf is revolved to a vertical position by pulling a trigger. The needles, left without support, drop into the water and are guided to their proper position by the turned shelf behind them and the needles already placed. The revolving shelves are then removed.

When a rise occurs in the river, some relief may be given by opening the gates of the lock and by pushing out the heads of alternate pins, supporting them by sticks 12 to 15 inches long, placed between them and the support bar. If these measures are insufficient to keep the water from rising, the needles of the pass or of the whole dam may have to be removed.

According to the original intention the needles were to be released in the usual manner by turning the jack-posts. It was feared, however, that owing to their weight the needles might get more or less injured when released, especially those falling over the weir. Another method of removing the needles was therefore adopted. Each needle is provided on top with a counter-sunk handle. A chain much longer than the dam is passed along the up-stream side of the needles and connected by hooks with their handles so as to have a considerable amount of slack chain between each pair. A long line is connected to the end of the chain. It can be wound up either

by the engine of the boat or by a crab on the lock-wall or on shore. As this line is wound up the needles are pulled in succession out of their place in the dam. This method works very rapidly and satisfactorily.

Drift-boom.—After severe storms the river carries a considerable amount of drift, which may injure the dam and interfere with its being lowered. To avoid this danger a drift-boom, consisting of four parallel timbers bolted rigidly together and having rudders at intervals of 30 feet, is placed across the river from a point some distance above the lock to the crib at the river-wall of the lock. As the river makes a sharp bend at the point of attachment the boom forms a continuation of the shore. It serves to guide the drift into the lock, where it can be held or let through as desired. The rudders are all controlled by a wire rope which connects them all and is wound on a capstan at the end of the boom. By setting the rudders at any desired angle the boom can be held out in the stream at any point required.

Cost.—The total cost of the dam, including the pass, weir, pier, and abutment, amounted to $73,697.74, or $245.66 per lineal foot. The substructure cost $226.48 and the superstructure $19.18 per foot.

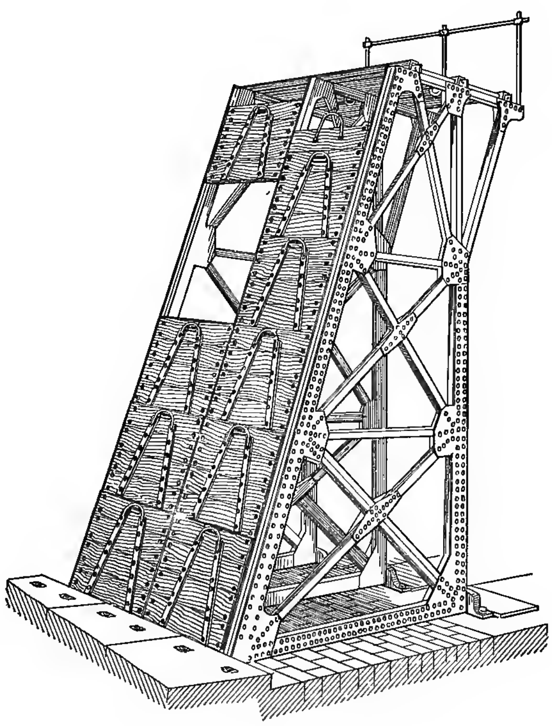

FIG. 40.—BOULÉ GATE.

Boulé Gates (Fig. 40).—In 1874 M. Boulé introduced a modification in a Poirée dam by substituting for the needles ordinary plank sluice-gates, a number of gates, placed one on top of another, being used in each bay. Each of these gates consists of a number of boards, tongued-and-grooved, and bolted together. They slide vertically between the frames and are maneuvered by a derrick travelling on top of the foot-bridge. In order to limit the transverse strains, to which the gates are subjected, the distance between the frames should not exceed one meter. Thicker boards are used for the lower gates than for

those placed at the top. For convenience in regulating the height of the pool, the upper gates may consist of single planks which can be readily placed or removed by hand.

While the first cost of a frame-dam with Boulé gates is about the same as that of a needle-dam, the former has the following advantages over the latter:

1st. It forms a tighter dam, as it has fewer joints.

2d. It can be more correctly proportioned to the water pressure to be resisted, thin planks being used for the upper and thick planks for the lower gates.

3d. It reduces the spans of the wooden members greatly, by placing them horizontally between the frames.

4th. It can be used for deeper pools.

5th. The service-bridge can be placed at a higher level above the water.

6th. No weir is required, as the whole dam forms an overflow.

7th. The level of the pool can be easily regulated.

8th. The dam is more easily maneuvered and with less danger.

On the other hand, it must be stated that a needle-dam can be opened much more rapidly than a Boulé dam, as constructed at present.

Compared with a dam of Chanoine wickets, Boulé's system is found to be considerably cheaper and less complicated.

By removing in succession each row of Boulé gates across the whole dam, the pool is lowered gradually and the work of raising the gates is greatly reduced. This method of maneuvering consumes, however, considerable time, 5 to 6 minutes being required for raising

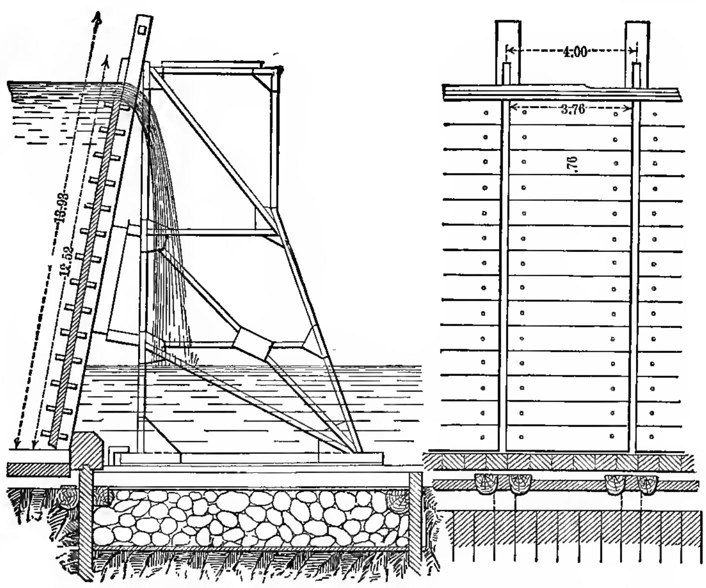

FIG. 41.—BOULÉ GATES IN MOSKOW DAMS.

one of the gates of the lowest tier. This objectionable feature might be removed by introducing some system of escapement.

Boulé gates were first used in France in the regulating portion of the Mulatière dam across the Saône, near Lyons. They have since been successfully used in the dams at Suresnes, Marly, etc.

This system was applied by M. Janicki, in 1876, in six dams on the river Moskowa, in Russia. According to the original plans, these dams were to be provided with needles 7

inches square. As the engineers had some doubts about being able to work needles of that size, they adopted Boulé's system, which had just been proposed, with some modifications. Instead of gates, planks about 10 inches wide are used, which bear against upright timbers resting against a sill and the top of the frame like Poirée needles (Fig. 41). Two pegs are put through every plank, one at each end. They bear on the down-stream side against

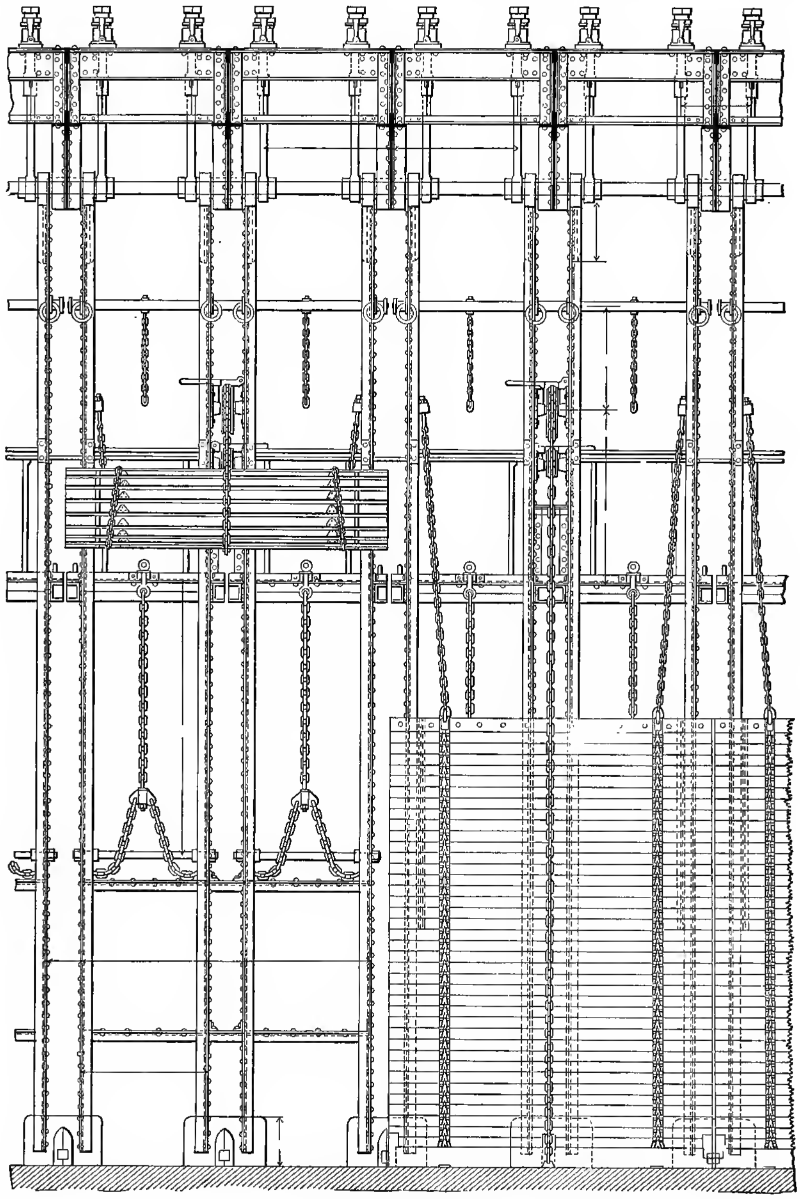

FIG. 42.—CAMÉRÉ CURTAIN-DAM.

the upright timbers, and serve as guides for the planks. On the up-stream side they form the handles by which the planks are raised, by means of hooked poles, no crab being required. The objection to the time consumed in maneuvering dams arranged according to Boulé's system applies also to the Moscow dams, but it would seem that in this case an arrangement for permitting the planks to escape might be readily contrived.

The Curtain-dam, invented by M. Caméré, was first introduced in 1876–80 in the Port

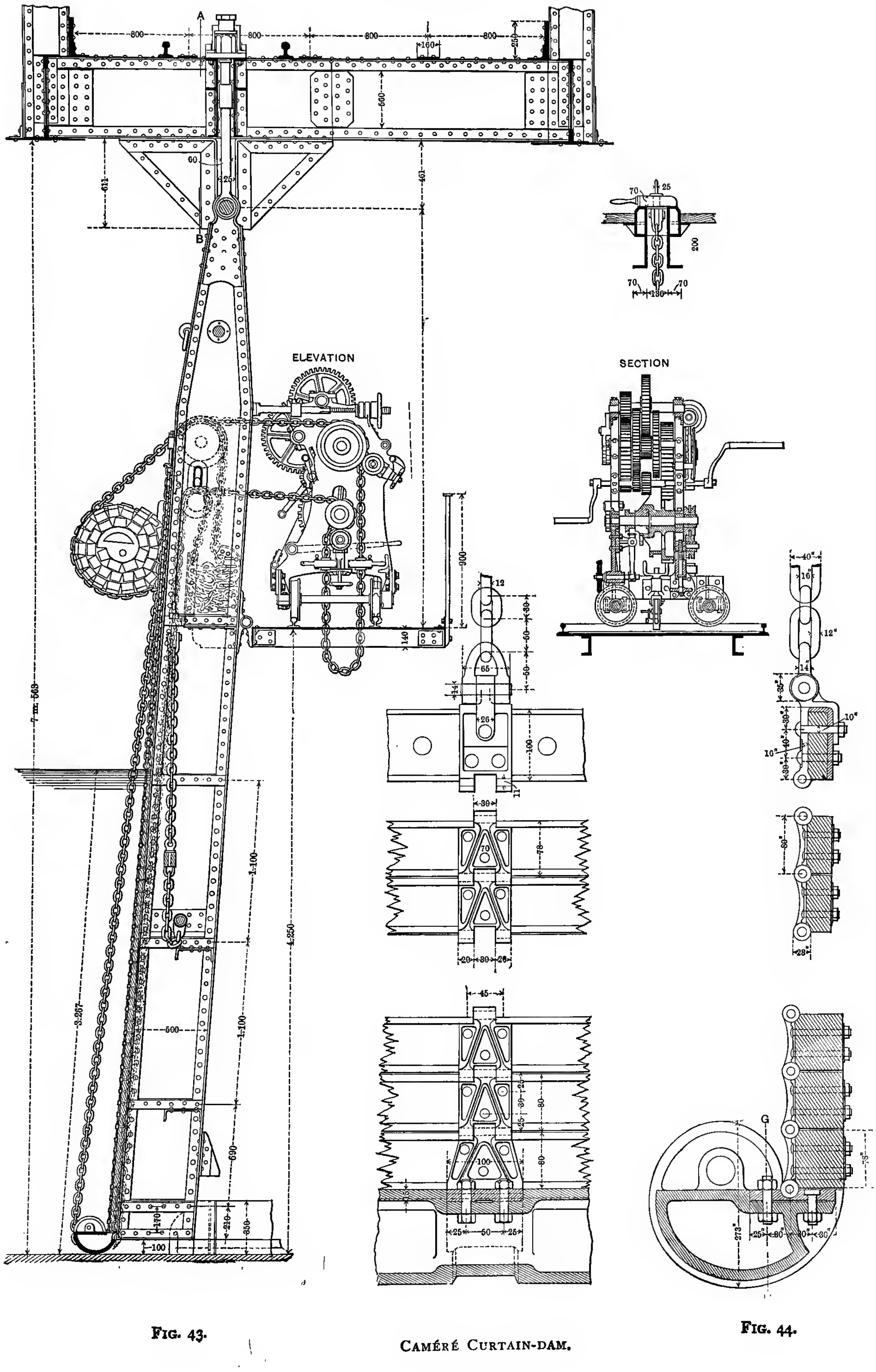

FIG. 43. FIG. 44.

CAMÉRÉ CURTAIN-DAM.

Villez dam (page 167). Having stood this test very successfully, it was used later on in the Suresnes, Poses, and Port-Mort dams (pages 167 to 171). M. Caméré's invention consists in the substitution of a wooden curtain that can be rolled up from the bottom, for the needles in a Poirée dam.

Figs. 42 to 46 show the construction of the curtains, etc., of the Poses dam,* which are called "double," as each of them covers two bays of the dam. In the other two dams mentioned above, a curtain is provided for every bay. Each curtain consists of a number of horizontal wooden bars, which are fastened together on the up-stream side by two rows of bronze hinges (Fig. 44). The bars have the same length and height, but their thickness is increased from the top to the bottom, according to the pressure they have to sustain. A casting called the "rolling-shoe" is attached to the bottom bar and forms the centre on which the curtain is rolled up. It rests on the floor when the curtain is down. The base of the shoe forms half the spire of an Archimedian spiral, which is completed by three

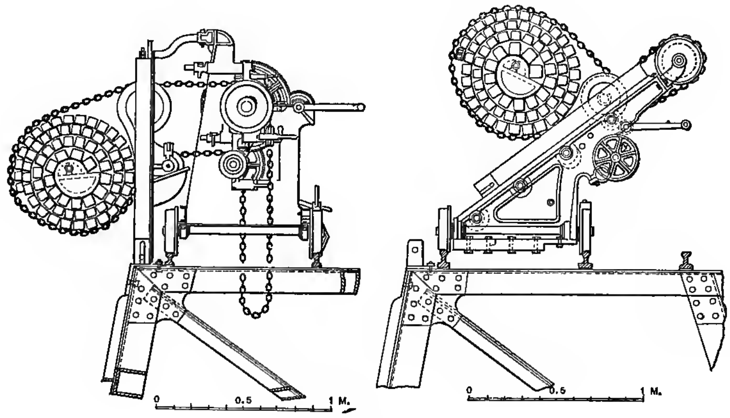

FIG. 45.

flanges which surmount the upper plane surface of the shoe. The weight of the "rolling-shoe" is sufficient to make the curtain unroll easily when it is being lowered.

The curtain is suspended by two chains which are fastened by hooks to the fixed parts of the dam, above the water. Each of these chains is attached to a ring bolted to the upper bar in the line of the hinges.

The curtain is moved by means of a special windlass (Fig. 45), which works an endless chain that passes around the curtain on its centre-line. The chain is prolonged above the curtain and is guided to the windlass by fixed pulleys. The windlass is arranged in such a manner that when the curtain is being rolled up, the up-stream part of the windlass-chain, which rises, travels faster than the down-stream part, which is lowered. This difference of velocity causes the chain to slide under the shoe. The resulting friction added to the traction of the chain makes the shoe revolve, and thus rolls up the curtain. In unrolling the curtain the down-stream part of the chain is made fast and the up-stream part is released. If the curtain is properly suspended it will move between two vertical planes. It is, however, advisable to have guides to prevent any lateral motion which might be caused

* Figs. 42 to 46 are taken from Dr. William Watson's official report to the U. S. Government on "Civil Engineering, Public Works, and Architecture at the Paris Universal Exposition of 1889."

by faulty construction or regulation. These guides are usually formed of angle-irons which are attached to the frames.

The hooks of the chains by which the curtain is suspended are fastened to a special iron frame (Fig. 45), which is secured to the bridge of the dam by pins. When the curtain is to be removed, after being rolled up, it is placed with the frame from which it is suspended, on a special car (Fig. 45) running on the bridge track. After being taken out of the dam, the curtains must be hung up to dry, and cleaned.

With the Caméré system of movable dam a special weir is not required, as no damage can result from the water passing over the top of the curtains. The upper

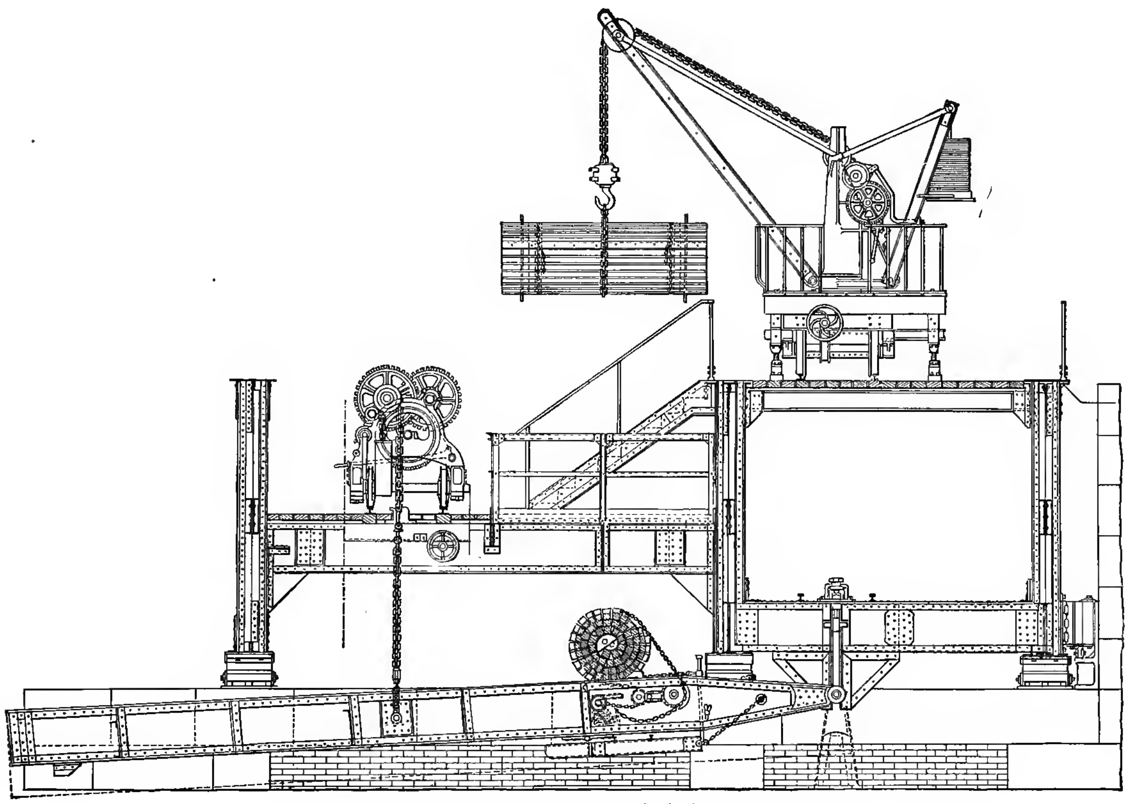

FIG. 46.—CROSS-SECTION OF CAMÉRÉ CURTAIN-DAM.

pool can be drawn down by rolling up the curtains to any desired height. As the water is thus discharged from the bottom of the pool, no difficulty is experienced with drift, but on the other hand, it involves the objectionable feature that scour is produced at the bottom of the curtains when they are raised. The Caméré curtains have now stood the practical test of fifteen to twenty years' service in the dams of Port Villez, Poses, Suresnes, and Port-Mort.

The Port Villez Dam was constructed in 1876–80 across the Seine at a point about 90 miles below Paris, to obtain a depth of 10½ feet for navigation. It is 700 feet long and consists of two central navigable passes and a regulating weir on the right bank, which are separated by two piers. The sill of the passes are 13.12 feet below the upper water-level; the sill of the weir is at half this depth.

The original plans contemplated the construction of a Poirée dam with needles 8 inches square, which were to be handled by mechanical means. Thus far this system had only been applied to lifts of about 6½ feet. As M. Caméré, the engineer who designed and constructed the works under the direction of M. Lagrené as Chief Engineer, had some hesitation of using needles for the great lift required in the Villez Dam, he invented a hinged wooden curtain (page 164), which he used with Poirée frames, both in the passes and in the weir.

The frames are placed 3 feet 7¼ inches apart. Those for the passes are 18 feet high and weigh each 4181 pounds. The weir frames are 9 feet 2 inches high, and weigh 798 pounds apiece. The frames are designed to present as little obstruction as possible when lowered. They lie in a recess in the masonry apron when down. The up-stream posts have a small "T-iron" on their face, the web of which serves as a guide for the bars of the curtains. The service-bridge is widened sufficiently to carry two tracks, by means of brackets on the down-stream side of the frames. The rails act as the braces for the frames and replace the connecting bars used in the older types of frames.

The frames are raised or lowered by means of a windlass which is placed on a car that travels on one of the tracks of the service-bridge. The lowering of the heavy frames of the passes is a troublesome operation. Numerous breakages and deformations of the frames, caused by their striking on stones, stumps, etc., brought down by the floods, have occurred.

The Poses Dam on the Seine, about 125 miles below Paris, was constructed to replace an older movable dam. The work was completed in 1885. The dam, which extends from the left bank of the river to the point of an island (Fig. 47), has seven

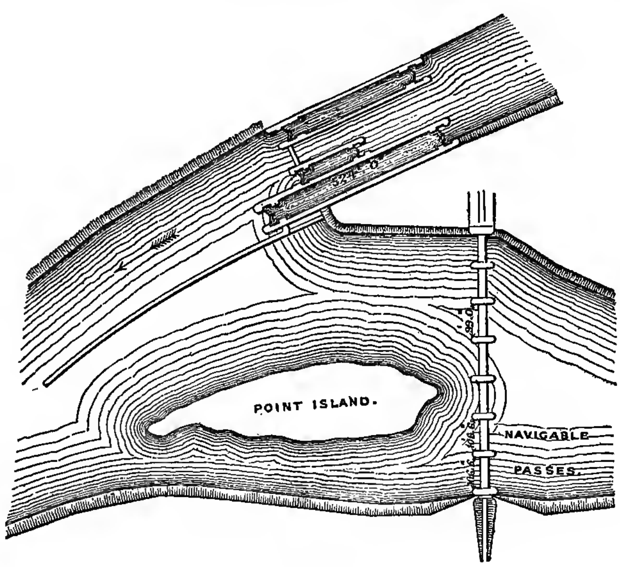

FIG. 47.—POSES DAM.

openings which are separated by piers. The two openings at the left bank, serving as navigable passes, are each 106½ feet wide; the others have each a width of 99 feet. The sills of the navigable passes and of the three passes nearest the island are 16.42 feet below the upper water-level. The two central openings have their sills 6⅝ feet

higher than those of the other passes. The locks are located between the island, at which the dam terminates, and the right bank of the river.

On account of the great height of the dam, the foundation across the whole river was carried down to an impermeable stratum of chalk, at a depth of about 28 feet below the sills of the navigable passes. The piers (Fig. 48) are 13.12 feet

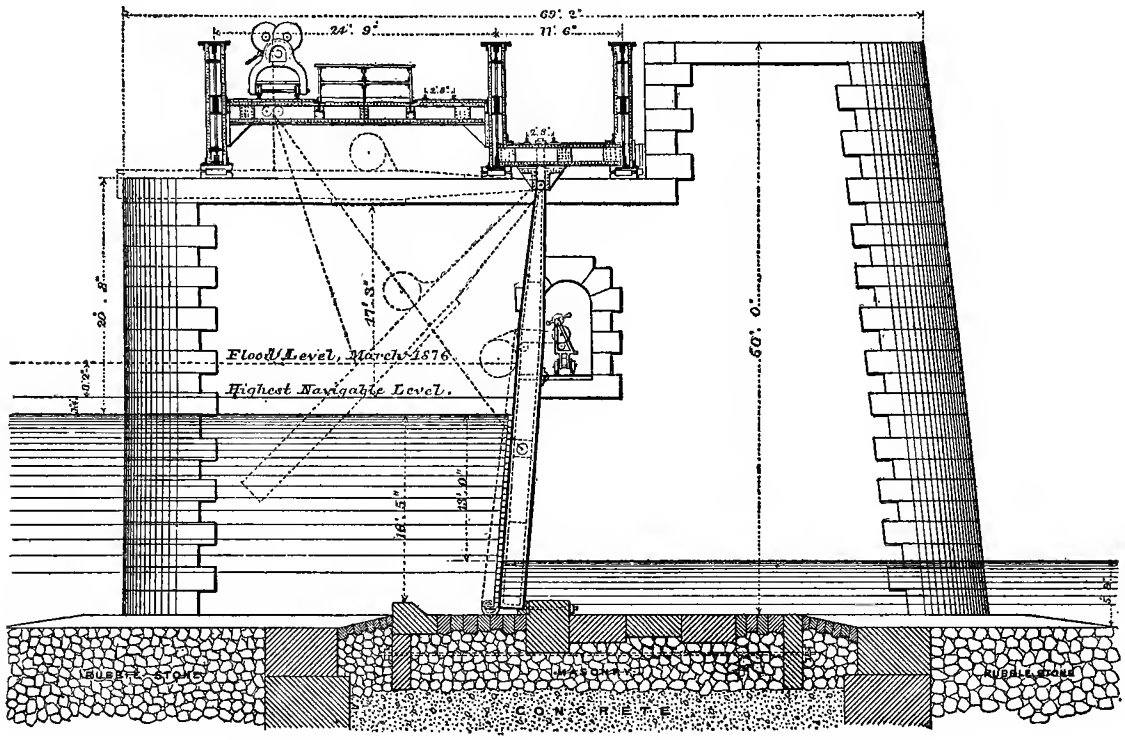

Fig. 48.—Pier of Poses Dam.

thick. Full-centred arches, 4.26 feet wide and 7.54 feet high, are constructed in the piers and abutments to permit the service-bridge to pass through them.

On account of the trouble experienced in handling the heavy Poirée frames of the Villez dam, mentioned above, M. Caméré, who designed and constructed also the Poses dam, decided to use in the latter, frames suspended from an overhead bridge. When down, these frames bear against a sill and form the support for the Caméré curtains. When the dam is to be opened, the curtains are first removed, and then the frames are hoisted out of the water, so as to lie in a horizontal position below the bridge. A similar arrangement of suspended frames was suggested by M. Tavernier for the Saône in 1873, but was not carried out. While this system has the advantage of removing the whole dam from the water when the passes are to be opened, it necessitates a high service-bridge and, consequently, long frames, if vessels are to pass under the bridge during floods.

A wide bridge is constructed across the whole dam. It is composed of three lines of longitudinal lattice girders connected by cross-girders. The longitudinal girders divide the bridge into two parts at different elevations (Fig. 46); one supporting the upper, suspended ends of the frames and carrying the derrick used in removing

the curtains; the other supporting the windlass for hoisting the frames out of the water.

Each curtain in this dam is 7.47 feet wide and closes two bays. It is composed of yellow-pine bars 0.25 feet high, having a slight play between them to allow for swelling. The thickness of the bars varies from 1.57 inches at the top to 3.54 inches at the bottom of the deep bays. The upper bar is reenforced by an angle-iron, as it is exposed to shocks from floating bodies. The curtains, when rolled up, are not removed from the frames unless they require repairs. Even when the frames are hoisted up, the curtains remain attached to them. In one of the deep passes a curtain can be rolled or unrolled in about fifteen minutes. The raising and lowering of a frame requires, respectively, twenty and ten minutes.

The Dam at Suresnes, a short distance below Paris, was originally constructed for Poirée needles. In 1884 it was reconstructed in order to raise the level of the pool $3\frac{1}{4}$ feet, to secure a minimum depth of $10\frac{1}{2}$ feet of water. In building the new dam M. Boulé, the Chief Engineer in charge of the work, decided to use both the gates invented by him and Caméré curtains, in order to test the two systems side by side. At the site of the dam two islands divide the river into three channels, Fig. 49. A weir 206 feet wide was

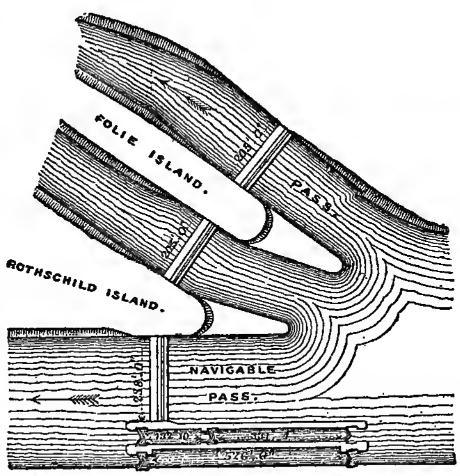

FIG. 49.—SURESNES DAM.

built in the middle channel, and passes respectively 206 feet and 238 feet wide were constructed in the right and left channels. The sills of the right pass, weir, and left pass were placed respectively 12.13, 16.25, and 17.90 feet below the level of the upper pool. A lock 525 feet long and 56 feet wide is constructed at the left bank.

The weir is closed by Boulé gates, Fig. 40, page 162. Caméré curtains are used in the right pass. In the left pass, which is the principal channel for navigation, Caméré curtains and Boulé gates alternate. The curtains and gates are supported by Poirée frames. Those of the main pass are 19.5 feet high and weigh each about 4000 pounds. Each of the curtains used in this pass weighs 1600 pounds.

The frames are maneuvered by means of a Megy patent windlass, which is placed on the abutment. A continuous chain unites the frames by means of link catches placed on their upper cross-braces. The portion of the chain between any two con-

secutive frames is longer than the distance between their centres. As the chain is wound up several frames are moved at a time, like the sticks of a fan. This system is similar but not as good as that used in the Louisa Dam, described on page 159. At Suresnes seven men can open a pass of 238 feet, containing 57 frames, in three hours and can raise it in five. At Louisa three men can raise a frame and place the foot-bridge in about a minute.

The Port-Mort Dam was constructed in 1886 across the right branch of the Seine, between the Port Villez and Poses dams. It has seven passes of 99 feet width, which are separated by piers 13 feet wide and are closed by Cameré curtains. The dam is very similar in design to that at Poses, except in some minor details of construction. Owing to the much greater height of the navigable level above the normal level, the piers of this dam are higher and its frames are longer than those of the Poses dam. In both of these dams a clear headway of 16½ feet above the highest navigable level is provided when the frames are raised.

CHAPTER II.

SHUTTER-DAMS.

Early Shutter-gates.—For some centuries gates turning on horizontal axles, placed near their tops, have been used in Holland to let the interior water from rivers and canals escape into the ocean at low tide while preventing the water from the sea from entering at high tide. Such a gate having its axle placed at one-third its height from its base, which corresponds to the centre of pressure when the water rises to the top of the gate, was proposed by M. de Cessart, in a "Description of Hydraulic Works," printed in 1808. M. Petitot suggested, in 1825, a similar gate for regulating automatically the level of a stream of water.* In a memoir on establishing internal navigation between Paris and Rouen, M. Frimot proposed, in 1827, placing in fixed dams several gates, one on top of the other, each turning on a horizontal axis. The turning of these gates was to be controlled by floats.

In 1837, shutters turning on horizontal axles were placed under the bridge of Riom (France).† These gates opened automatically during freshets and had to be set up again by hand.

The first dam constructed by movable shutters was probably the one across the river Orb (France), described by Delalande in his "Traité des Travaux de Navigation," published in 1778. This dam was raised 3 feet by movable wooden shutters hinged to the top of the dam and held by props placed on their down-stream side. The sluice-openings were closed by stop-planks (poutrelles) placed horizontally, one on top of another, and attached to each other by chains. When the water was to be lowered these stop-planks were allowed to escape by a suitable contrivance and the shutters on top of the dam were lowered by hand when the water had subsided sufficiently.

Thénard Shutters.—M. Thénard applied the movable shutters just described, with some improvements, on several dams on the river l'Isle (France). He found that the height of these dams, 6.56 feet above low water, while insufficient for internal navigation, caused inundations in times of freshets. To avoid this trouble M. Thénard determined to reduce the height of the fixed dam to 3.93 feet above low water, and to obtain the remaining height required for navigation by means of movable shutters similar to those placed on the crest of some of the dams in the river Ord. He used this constructian for the first time in 1831 in the dam of Saint-Seurin. The principal improvement which M. Thénard introduced was a tripping-bar, by means of which the props could be "tripped" in succession, allowing the shutters to fall down. The end of this bar consisted of a rack, which could be moved by a pinion placed in a well in the abutment.

* Mémorial du Génie, for 1825, Vol. VII., page 161.

† Annales des Ponts et Chaussées, for 1842, 1st Series, page 231.

While the device just described made the lowering of the dam a very simple operation, much difficulty was encountered in raising it against the current. To obviate this trouble, M. Mesnager advised M. Thénard to place counter-shutters falling up-stream above the remaining shutters. The former were to be raised by the current itself and to be kept by stop-chains from rising too high. This plan was carried out successfully on three dams built on the l'Isle in 1839–1841. Each of these dams was 230 feet long, including the pass and lock. The shutters were bolted to a wooden sill which was fastened to the top of the masonry dam. They were 6.56 feet wide by 3.28 feet high. Those falling down-stream were supported, when up, by wrought-iron props, whose feet bore against iron hurters (shoes) or sills. The iron tripping-bar was moved by turning a pinion on the river bank in the manner already explained. It had a projection for every prop. For every $1\frac{1}{8}$ inches of motion it tripped one shutter. After all the shutters were lowered the tripping-bar had to be moved back to its original position.

The up-stream shutters, when lowered, were held in place by spring-latches, which could be released by a tripping-bar. A forked chain, fastened to the floor, kept each shutter from rising above a desired point. When these shutters were up the lock-keeper could walk almost dry-shod on the weir and raise the down-stream shutters by hand. When both sets of shutters were up he equalized the level of water between them by opening small valves in the up-stream shutters. He finally pushed the counter-shutters down with a pole. The whole operation of raising one of these dams could be performed by one man in about sixteen minutes.

In 1843, M. Thénard erected at the St. Antoine Dam shutters 5.57 feet high by 3.82 feet wide. In this case he constructed a small sheet-iron foot-bridge on top of the counter-shutters, from which the tender could raise the down-stream shutters.

Having applied movable shutters 5.57 feet high successfully, M. Thénard obtained, in 1846, authority to carry out his system on the Seine near Montereau. According to a memoir which he prepared he intended, in this case, to use shutters 7.54 feet high by 5.08 feet wide, and counter-shutters 7.04 feet high by 5.02 feet wide. The apron was to have a length of 36 feet. Some improvements were to be introduced in the manner of releasing and lowering the counter-shutters. The main shutters were to be raised from a boat. Before he could carry out his project M. Thénard was put on the retired list. His successor, M. Chanoine, introduced several modifications before such a dam was actually built in 1850 across the Seine at Courbeton.

Although Thénard's system of movable dams has been used successfully, the counter-shutters form rather an objectionable feature. The stop-chains, their attachments, and the hinges of these shutters are subjected to great strains when the shutters are suddenly stopped at the proper height. Breakage of these parts at any one shutter delays the erection of the whole dam. It has also been found difficult to make the temporary dam, formed by the counter-shutters, sufficiently tight.

Thénard shutters have been used, with some modifications and improvements, on dams in India.* In the Mahanuddee Dam (Fig. 50) ten openings, 50 feet in width,

* "Movable Dams in Indian Weirs," by R. B. Buckley, Minutes of Proc. Inst. C. E. for 1880, Vol. LX., p. 44. Fig. 50 is taken from this paper.

are each closed by seven pairs of shutters, those falling down-stream being 9 feet high. In the Sone Dam sixty-six openings, 20½ feet wide, are each closed by a single pair of shutters, those on the down-stream side being 9½ feet high. To regulate the rising of the counter-shutters, and to avoid the heavy shocks to which they would be subjected if stopped suddenly, hydraulic brakes are attached to their up-stream side. Each of these brakes consists of a cylinder full of water, in which a piston, moved by the shutter in rising, travels. A number of small escape orifices for the water are made in the

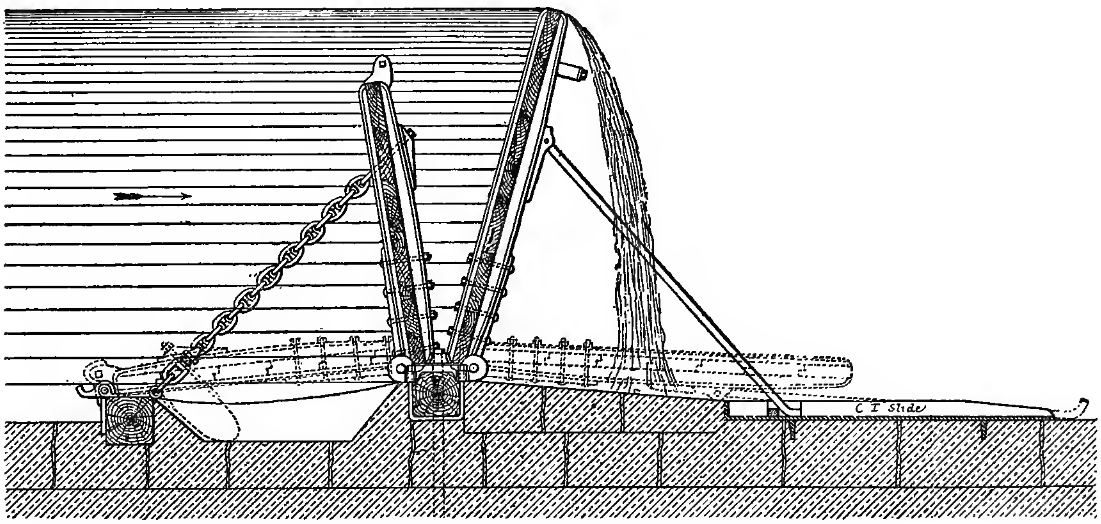

FIG. 50.—THÉNARD SHUTTER-DAM.

cylinder and arranged in such a manner that the vent of the water is diminished as the piston rises, and the checking force of the brake, therefore, is increased.

Chanoine Wicket-dam.—The objection to counter-shutters mentioned above induced M. Chanoine to substitute for them in the Courbeton Dam across the Seine a Poirée needle-dam, which serves to hold back the water while the shutters on the weir are being raised and furnishes a bridge from which this operation can be performed. There still remained, however, the difficulty of raising the last shutter on the weir on account of the leakage through the needle-dam. In 1852, M. Chanoine overcame this difficulty by raising the axle of the shutter to a point between one-third and one-half of its height, and supporting it on a horse or trestle which itself could revolve on an axle fixed on an apron of the dam when the prop was withdrawn. This invention was carried out practically for the first time in 1857 in the dam of Conflans, on the Seine. Another engineer, M. Carro, appears to have invented a similar shutter about the same time.

The term "wicket" has been applied in the United States to a shutter revolving on an axle placed near its middle. Described in detail, a Chanoine wicket consists of three parts (Figs. 51, 52, and 53): A rectangular panel of wood or iron; the horse, or trestle, supporting the axle of the shutter, and the prop holding up the horse and having its foot bearing against a cast-iron shoe, called a "hurter" (in French "heurtoir") fixed to the apron. The parts of the shutter above and below the axle

are called respectively the "chase" and the "breech." Two maneuvering chains are usually attached to the shutter, one to the top and the other to the bottom.

Several wickets, placed side by side, form the dam. To prevent the panels from interfering with each other by swelling, etc., they are placed about 2 to 4 inches

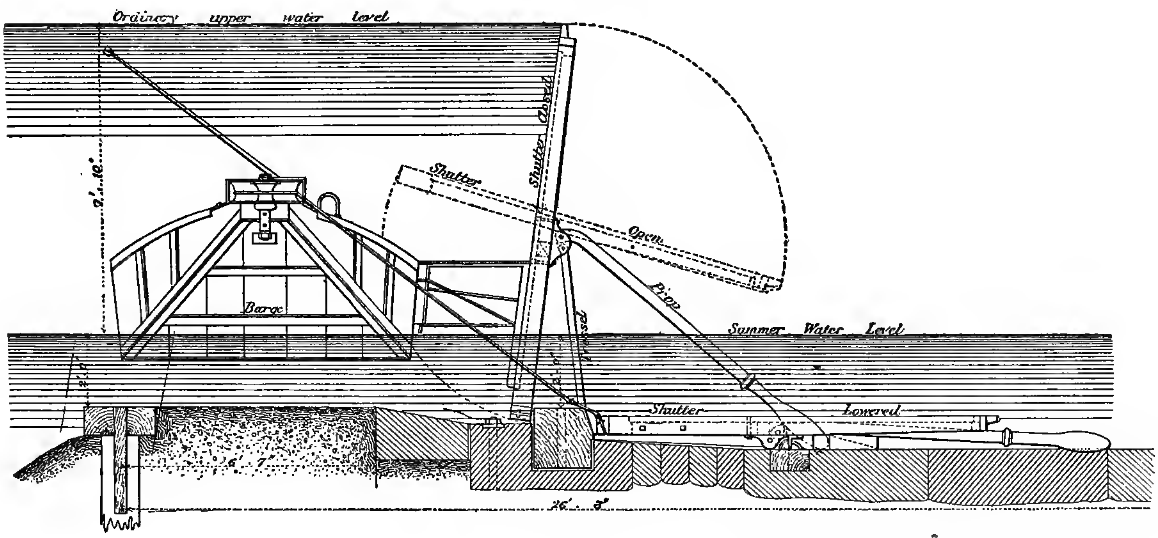

FIG. 51.—CHANOINE SHUTTER-DAM ON NAVIGABLE PASS ON THE UPPER SEINE.

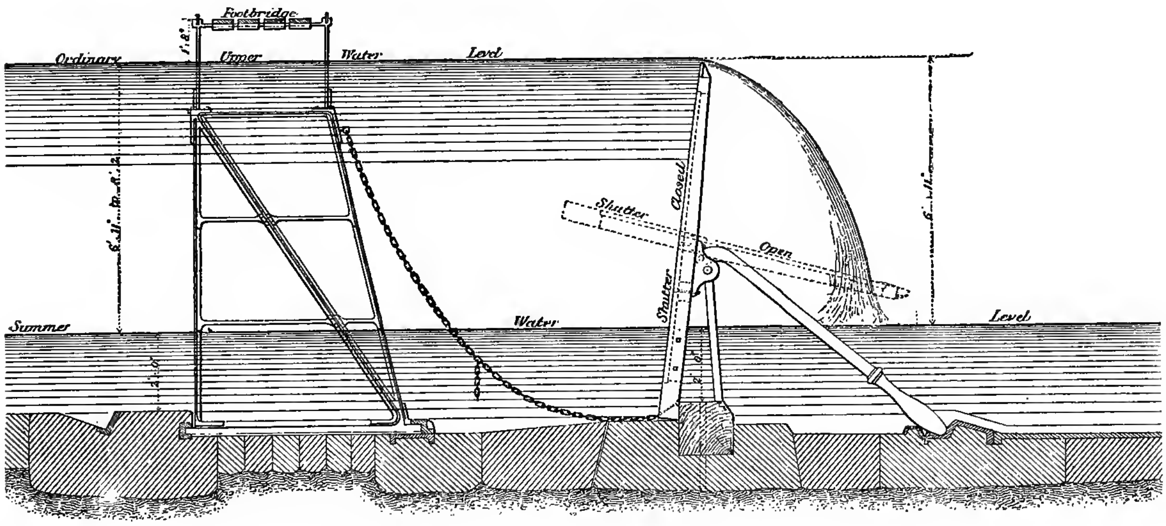

FIG. 52.—CHANOINE SHUTTER-DAM ON NAVIGABLE PASS ON THE MARNE.

apart. When the leakage between the wickets is greater than the minimum flow in the river the spaces between the wickets can be closed by needles or by nailing strips of wood to the shutters.

The axles of the weir-wickets are placed so that they will revolve automatically when the water reaches a certain height, but those of the pass are attached at the centre of the shutter. The pass wickets do not oscillate, therefore, when the water rises. They can be readily lowered by the tripping-bar when required. When the prop is tripped by moving it sideways, it slides forward on a casting joined to the hurter, called the slide, and is

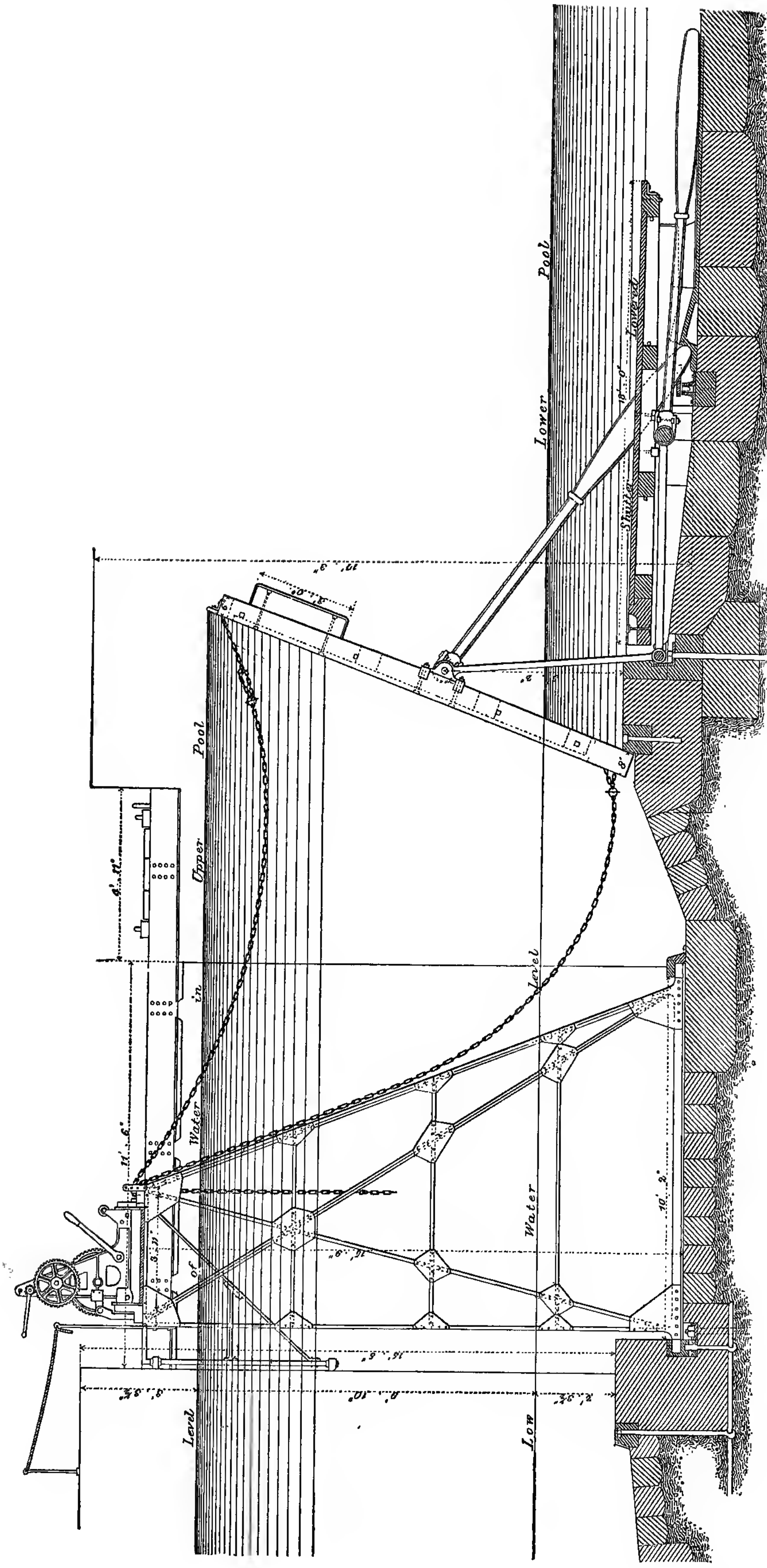

FIG. 53.—CHANOINE SHUTTER-DAM FOR NAVIGABLE PASS OF PORT A L'ANGLAIS WEIR ON THE UPPER SEINE (CONSTRUCTED IN 1870).

followed by the horse. The shutter falls on top and covers them. This method can only be used, of course, when there is sufficient water on the apron to act as a cushion. If the water on the apron is not deep enough for this purpose the fall of the wicket must be broken by means of the maneuvering chains.

The tripping-bar moves horizontally on the apron. It has a projection for each prop. They are placed in such a manner that at first only one prop is tripped at a time, then two, and finally three or four. By this arrangement the pool is drawn down gradually and the effort required in moving the tripping-bar is reduced.

As the tripping-bar must be pulled back to its original position when the dam is down, it must either be placed in a special channel on the apron or the tops of the props must be curved at the top as shown (Fig. 52), in order not to interfere with the tripping-bar when they are down. The tripping-bar is sometimes placed on rollers to facilitate its motion. One end of the tripping-bar is formed by a rack which engages with a pinion fixed on a vertical shaft and placed in a well built in the masonry of the abutment.

To raise the dam each wicket is brought to a horizontal position, or, as it is called, "put on the swing," by pulling the breech-chain from the service-bridge or a boat until the horse is up and the prop has fallen into its resting-step. If the breech-chain is pulled up too much there is some difficulty in afterwards raising the wicket to its vertical position. To avoid this trouble lugs are sometimes cast on the horse to confine the position of the wicket within 15° of the horizontal, when on the swing. When a foot-bridge is provided there is, however, no difficulty in raising the wickets by means of the two maneuvering chains and the lugs on the horse are omitted, as they have some disadvantages. After the wickets have all been brought to the swing, in which position they offer very little resistance to the current, they are rapidly raised by pulling the chase-chains until the wickets strike against the sill. When in position in the dam they are inclined about 20° from a vertical plane. Movable counter-weights are sometimes attached to the wickets to assist in raising them.

In some Chanoine wicket-dams no service-bridge is provided, the maneuvering being done entirely from a boat. In others a foot-bridge is used only for the weir, the pass-wickets being maneuvered by means of a boat. In each of these cases the details of maneuvering differ slightly from the manner we have described. Thus, when a boat is used for the weir and the pass, the wickets of the former are first put on the swing, but those of the latter are erected at once. In the dams built in 1860 on the upper Seine and Yonne a boat was used for the pass and weir, but later on, in 1869, foot-bridges were constructed for the weirs.

The ordinary tripping-bar of the Chanoine wicket-dam can only be operated for a certain length of pass. By using two of these bars, one operated from each side of the pass, this length can be doubled. While having the advantage of lowering a dam very rapidly when all goes well, any obstruction that might get in the channel of the tripping-bar or in front of a hurter would prevent the lowering of the dam. For this reason M. Pasqueau, in constructing a movable dam across the Saône near Lyons (page 178), decided to dispense with the tripping-bar and to arrange the wickets so that each could be lowered separately. He accomplished this by devising a special double-grooved hurter and slide (Fig. 54), in which a step *b* is placed in front of the step *a* on which the prop rests. When a wicket is to be lowered it is first put on the swing. The breech-chain is then pulled.

This drags the prop from its rest and lets its foot fall to the lower step *b*. As the face of this step is inclined so as to offer no support, the prop slides down a passage in the hurter and down "the slide" behind the hurter until the wicket has been lowered.

Chanoine wickets, having their axis of rotation placed at $\frac{1}{3}$ to $\frac{1}{2}$ their height, at the centre of pressure of the water, open freely when the water rises above their crest, but do not close rapidly enough when the level of the upper pool falls, and cause thus a loss of water. This trouble has been removed and the raising of the

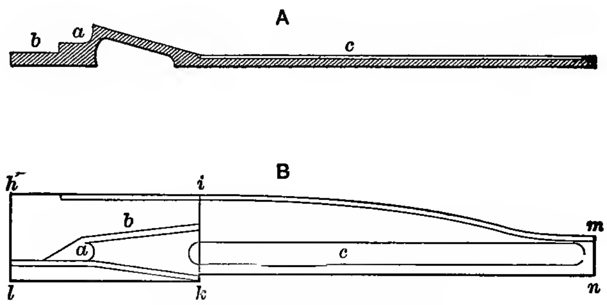

FIG. 54.—PASQUEAU HURTER.

shutters facilitated by placing one or two butterfly (flutter) valves near the top of the shutter (page 182). These valves (which are diminutive shutters about 3 feet high by 2 feet wide, revolving on horizontal axles) open when the shutter is being raised, and thus diminish the resistance. They regulate the level of the upper pool by opening and closing in a much more rapid manner for small variations than can be done by allowing the whole shutter to oscillate.

M. Chanoine's system has been largely used for movable dams on the upper Seine, Loire, Saône and Meuse. While it is more complicated and expensive than the systems of frame-dams which we have described, it has the advantage of permitting a much more rapid opening of the dam. When used with a maneuvering-boat this system is applicable to rivers carrying much drift, where any system of frame-dam would fail. It is for this reason that Chanoine shutters were used in the movable dams constructed on the Kanawha and Ohio rivers in the United States.

La Mulatière Dam, near Lyons, was constructed in 1879–81 across the Saône River at its junction with the Rhone. As the latter is a torrential stream subject to rapid rises and falls and often forces back-water into the mouth of the Saône, the dam was in the peculiar position of being exposed to sudden rises as well from below as from above. A sudden fall in the Rhone necessitated the rapid erection of the dam to maintain the required depth in the Saône. The dam had, therefore, to be constructed so that it could be rapidly raised or lowered.

To meet these conditions M. Pasqueau, the engineer in charge of the work, introduced various modifications in the system of Chanoine wickets, which was used, by inventing new details for the construction and devising new methods of maneuvering.

As a great deal of gravel was carried by the stream, M. Pasqueau dispensed with the tripping-bar, which might have easily been obstructed, and arranged the wickets to be raised or lowered from a maneuvering-bridge. This was made possible by using a double-stepped hurter or resting-shoe for the props, described on

page 178. The ordinary tripping-bar cannot be used for passes much over 150 feet wide. With Pasqueau hurters the width of the pass need not be considered. In the Mulatière dam it is 340 feet wide, no piers being placed in the stream.

Iron wickets are used in this dam, as those of wood only last for ten years. The panels are formed of two "U" irons, 2.95 feet apart, covered by $\frac{3}{16}$-inch plate iron, projecting 10 inches beyond the uprights, supported by braces, and having angle irons put on the edges. They are 14.3 feet long and 4.6 feet wide. A flutter-valve, 5 feet by 3 feet, is put into the upper half of each wicket. It is held in place by a bell-crank and is operated from the bridge by means of a pole.

The dam is maneuvered from a Poirée service-bridge by means of a steam windlass. The frames, which are 22.3 feet high, are placed 9.8 feet apart—more than twice the usual distance. As a frame when down laps only over two other frames, the bedding trench is only 28 inches deep instead of the four feet usually required. The frames are symmetrically built in the form of a double St. Andrew's cross, possessing great stiffness. They are said not to silt up as much as those having vertical up-stream posts. Instead of the old style axles, they have at the bottom pin connections with the journal boxes in the floor.

The Dams on the Great Kanawha River* are of the Chanoine-wicket type, and are operated from frame service-bridges. They are the first movable dams erected in the United States for slack-water navigation. The first two of these dams were completed in 1880, and others have since been erected with improvements in the details of construction. As their general features are very similar it will suffice to describe one of these dams.

Dam No. 7 (Fig. 55), which was completed in 1892, is located about forty-four miles above the mouth of the river. It has a pass 248 feet wide, a weir 316 feet wide, and a lock 342 feet long between quoins, with a width of 55 feet in the clear. A masonry pier 10 feet wide and 34.6 feet long separates the pass and weir. The dam terminates at one bank at the lock-wall and at the other at a masonry abutment.

The foundation of the pass is 50 feet wide. It was prepared by laying a bed of concrete on solid rock or hard-pan. The timbers to which the wickets are bolted are placed on this concrete. The wooden sill against which the wickets bear, when up, was laid 2 feet below low water. The edges of the foundation are coped with large bush-hammered stones secured by bolts.

The pass is closed by sixty-two Chanoine wickets having Pasqueau hurters. The wickets are of oak with pine panels and are banded with iron. They are placed 4 feet apart, and are 3 feet 9 inches wide and 14 feet $\frac{1}{8}$ inch long. The 3-inch space between the wickets can be closed with scantling when required. The axis of rotation is placed 6 feet 10 inches from the bottom of the wicket and 5 feet 11 inches vertically above the top of the sill. When up, the wickets form an angle of 20°, with a vertical plane and lap five inches on the sill.

* The account given of these dams is condensed from a description written in 1892 by Mr. Addison Scott, the Resident Engineer in charge of the work. This description was published by the Government, and is given also in W. M. Patton's Treatise on Civil Engineering, pp. 1494-1507.

The service-bridge of the pass is formed by thirty wrought-iron frames, each having a section of the floor and connecting-rods for joining it to the next frame attached. The frames are 8 feet apart between centres, and are connected by the chain used in raising them. When erected the bridge floor is 2½ feet above the top of the wicket, which is the normal pool level.

The weir is closed by seventy-nine wickets set 4 feet apart, each being 3 feet 9 inches wide and 9 feet 2½ inches long. The axis of rotation is placed 4 feet from the bottom of the wicket and 3 feet 4⅝ inches vertically above the top of the sill. The wickets make the same angle with the vertical as those of the pass, and lap 4 inches

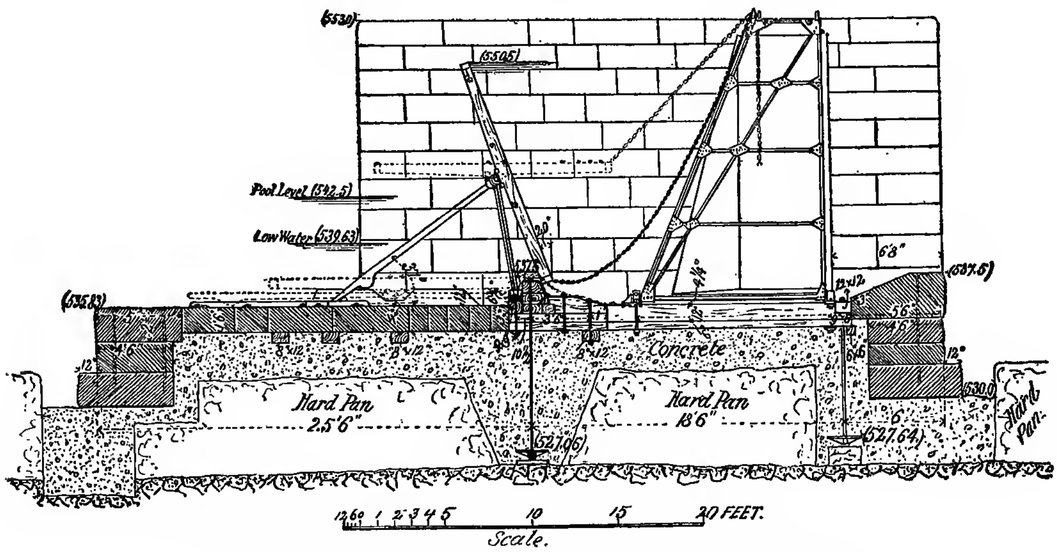

FIG. 55.—SECTION OF THE NAVIGATION PASS OF THE KANAWHA DAM.

on the sill when up. The service-bridge of the weir is similar to that of the pass. The wicket sill is of cast-iron. It is made in sections and has the horse-boxes attached.

The dams are maneuvered in the usual manner from the service-bridges. A light service-boat, having a derrick and capstan, is kept at each of the dams to assist in the maneuvers. The different dams and the central office in Charleston are connected by telephone so that due notice can be received of any rise in the river, etc.

Each of the dams can be erected by four or five men in seven to twelve hours, eight hours being usually required. This force can lower the dam in about two hours. Four men are constantly employed at each dam, one or two extra men being engaged when needed. The cost of operating and maintaining one of the dams amounts to about $2500 a year. A certain number of buildings, such as a storehouse, a blacksmith-shop, a carpenter-shop, etc., are built for each dam.

Ohio River Dams.*—The Chanoine Dam at Davis Island, five and one-half miles below Pittsburg, is the first of a series of movable dams which are to be constructed to improve navigation on the Ohio River. The work on this dam was

*Engineering News, May 15, 1886, and Scientific American Supplement, August 1, 1891.

begun in 1878 and finished in 1885. A channel 456 feet wide, between the island and the south bank of the river, was closed by a permanent dam. A movable dam was constructed across the main channel of the river. This dam consisted originally of a navigation pass 559 feet wide, and of three weirs, respectively, 224, 224, and 216 feet wide. Later on, the first weir was shortened by the construction of a drift-gap 52 feet wide, closed by a bear-trap gate (page 192), and the pier between this weir and the pass was removed, widening the latter to 716 feet. A lock, 600 feet long between gates and 110 feet wide, was built on the north side of the movable dam. It is the largest and widest lock thus far constructed. The gates of this lock, instead of swinging in the usual manner, are rolled into recesses or slips built for them in the bank for a distance of 120 feet.

The movable dam between the lock-wall and the abutment at Davis Island has a length of 1223 feet. It is formed by 305 wickets, which are placed 4 feet apart, between centres. The wickets are made of oak. They are 3 feet 9 inches wide. Their length varies from 12 feet 11 inches in the pass to 9 feet 9 inches in the weir nearest the island. When erected the wickets are inclined down-stream so as to make an angle of 20° with a vertical plane. The 3-inch space between any two adjoining wickets can be closed by scantling when required.

The apron of the dam, upon which the wickets are placed, is a framed structure composed principally of 12″ × 12″ white-oak timbers, which are embedded in concrete. The wickets are anchored to this structure by long bolts. Cast-iron journal-boxes for the axles of the horses and Pasqueau hurters for the props are also bolted to the apron-timbers.

The wickets of the pass are maneuvered from a steel boat, having a winch at its centre. A pole is attached to the end of the rope from the winch. This pole is provided with a hook and serves for grappling the wickets when they are down. In lowering the dam, each wicket is caught on top and pulled up-stream until the props drops to the lower step of the hurter, and moves down the slide, taking the horse and shutter with it.

The wickets of the weirs are maneuvered from a Poirée service-bridge having frames 15 feet 1½ inches high, placed 8 feet apart.

A second Chanoine wicket-dam, like the one described above, is now (1899) being built across the Ohio River, about 25 miles below Pittsburg, Pa.

The Pontoon-dam invented by M. Krantz was to consist of a number of revolving shutters, each being moved by a pontoon which could be made to float or sink in a conduit constructed on the down-stream side of the dam. The pontoon was to be connected to the shutter by a hinge. It was to be fastened to the down-stream side of the conduit in which it was placed by another hinge, so that both the pontoon and shutter would revolve on horizontal axes.

M. Krantz gave, in 1868, a description of a design he had prepared for such a dam which was to retain 10 feet of water (Figs. 56 and 57)*. The shutters were to be 9.83 feet wide, 16.33 feet high, and to incline 30° from a vertical plane when up. They

* This description is given in full in Lagrené's Cours de Navigation Intérieure, Vol. VIII., p. 332.

were to abut against a sill and to have their axis of rotation 1.33 feet above the centre of pressure. Three flutter-valves (butterfly valves) (Fig. 58), 3.12 feet high and 2.04 feet

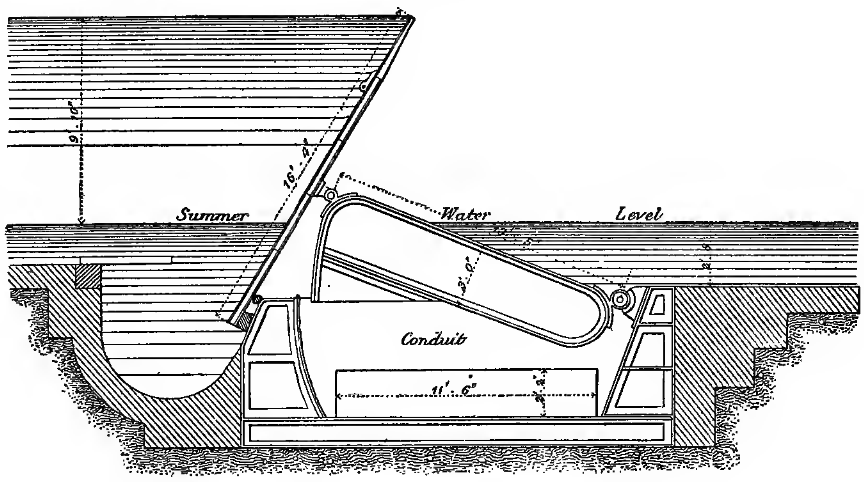

FIG. 56.—KRANTZ PONTOON-DAM.

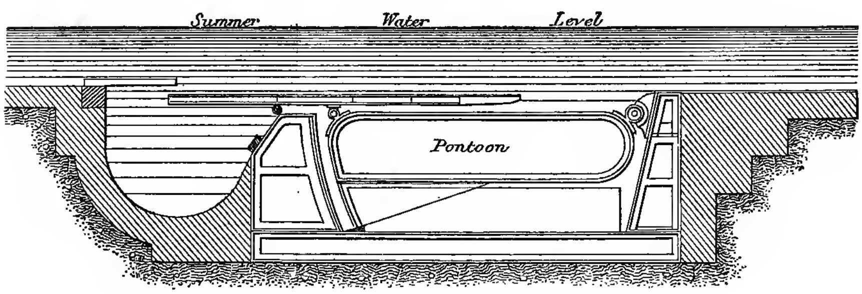

FIG. 57.—KRANTZ PONTOON-DAM.

wide, turning on horizontal axles, were to be placed near the top of each shutter to regulate the level of the upper pool when the dam was up. These valves were to work automatically. When the water in the upper pool should reach a certain level the valves were to open, but their motion was to be limited by stay-chains attached to the shutter so that the valves would not be at a greater inclination than 15° from the horizontal when open. If the level in the pool should fall, counter-weights fastened to the valves by chains were to close them.

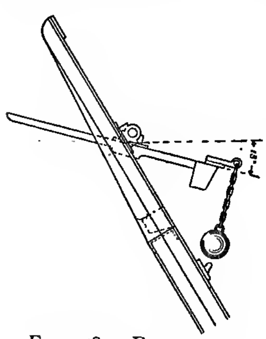

FIG. 58.—BUTTERFLY VALVE.

The pontoons were to be made of sheet iron. They were to have a rectangular cross-section and to be hollow. The manner in which they were to be hinged both to the conduit in which they were to be placed and to the shutters is shown in Fig. 56.

The conduit was to be made of cast-iron frames, which were to be surrounded by masonry. A small reservoir or lock, made of iron, was to be constructed at each end of the conduit. It was to be arranged in such a manner that it could be connected

by opening suitable valves either with the upper or lower pool. As these reservoirs were to be connected by openings with the conduit, the level of the water in the conduit could be varied by means of the valves of the reservoirs. By opening the up-stream and closing the down-stream valve the water from the upper pool would be admitted into the conduit and the pontoons would float and raise the dam. If the position of the valves were reversed, the water from the conduit would flow to the lower pool, and the pontoons would consequently sink and open the dam. By partly opening both the up-stream and down-stream valves the dam could be made to assume some intermediate position between the two extremes shown in Figs. 56 and 57. It was calculated that each pontoon in rising would have a sufficient force to raise a shutter.

The dam at Port Villez (page 167) was to be constructed according to the system of M. Krantz, but experiments conducted on a large scale at the site of a proposed lock at Bougival showed that a sufficient amount of water to raise a pontoon dam could not be drawn from the upper pool without lowering its level too much. This system was abandoned in favor of one invented by M. Caméré.

The Girard Shutter-dam (Fig. 59) was invented in 1869 by M. Girard, a prominent French engineer. It is a modification of the Thénard system consisting in raising

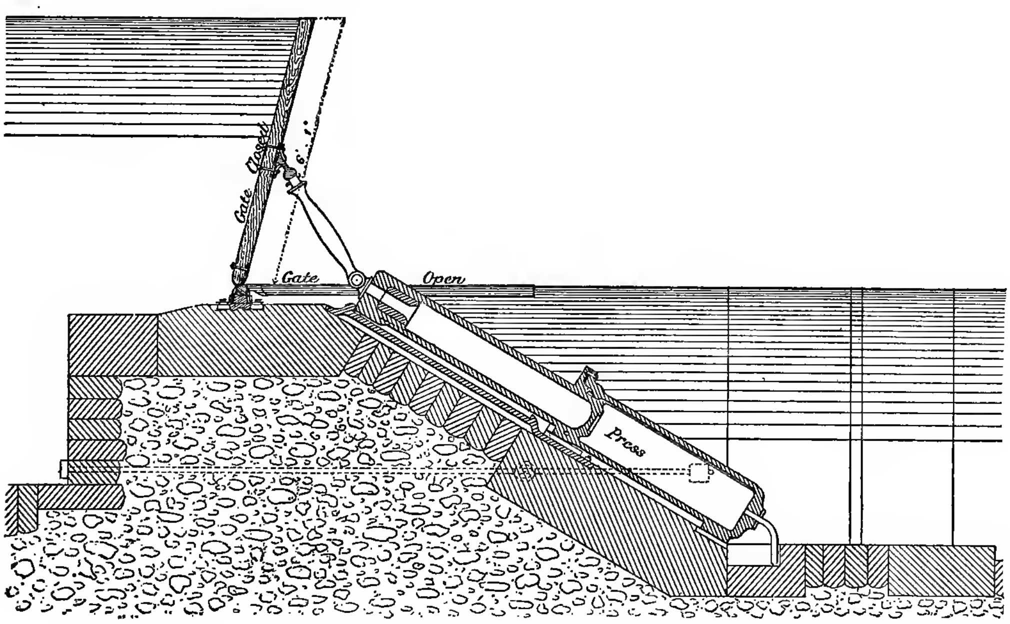

FIG. 59.—GIRARD SHUTTER-DAM.

the shutters by the pressure optained by means of hydraulic jacks placed on the apron of the dam.

Ile Brulée Dam.*—M. Girard obtained a contract to erect seven shutters according to his system on the weir of the Ile Brulée dam at Auxerre on the Yonne, the pass of which is closed by Chanoine wickets. He was killed in the Franco-Prussian war before he could finish the work. It was successfully completed by M. Callon. The shutters

* Annales des Ponts et Chaussées for 1873, 5th Series, Vol. VI, page 360.

on this weir are 11.55 feet wide by 6.46 feet high. Each shutter is formed of three pieces of "I" beams, which are covered on the up-stream side by fir joists 4 inches thick and on the down-stream side by sheet iron. The "I" beams are connected at the bottom to the axle on which the shutter turns. This axle is placed in a cast-iron hollow quoin, which is embedded in the masonry on the crest of the dam. Each shutter weighs 2552 pounds. A space of $1\frac{3}{4}$ inches is left between consecutive shutters.

Each shutter has a separate hydraulic jack, which is fastened to the down-stream side of the masonry apron at an inclination of 30°. The jacks are made of cast iron They have an exterior diameter of 16 inches and walls $1\frac{1}{2}$ inches thick. The piston is made of cast iron and is covered with a jacket of red copper. It moves a cast-iron cross-head four feet long, which is connected to the shutter by three rods attached to an axle placed on the shutter at about half its height.

A turbine water-wheel, 4 feet in diameter, operated by the head of water above the weir, furnishes the power for the hydraulic jacks. It is placed in an engine-house built on the abutment of the dam, and works both a water-pump and an air-pump. The water may be pumped directly to the jacks, but ordinarily it is pumped into a receiver in the engine-house, called an accumulator, having an inner diameter of 26 inches and a height of $11\frac{1}{2}$ feet. The accumulator is made of cast-iron and has walls 2 inches thick. It is connected with each jack by a copper pipe one inch in diameter. A three-way cock placed in the engine-house on each of these lines of pipe controls the communication between the pump, accumulator, and jack.

The accumulator acts as an air-chamber for the pumps, and stores sufficient power for raising the shutters when the upper pool is too low to turn the turbine-wheel. When it is empty, and the weir and pass are both open, the turbine-wheel is started by raising the Chanoine wickets in the pass by means of a maneuvering-boat.

As a rule, the Girard shutters are worked from the accumulator, and not directly from the pump. When the accumulator is empty, air is forced into it from the air-pump until a pressure of about 10 atmospheres has been obtained. Water is then pumped into the accumulator until the pressure has been raised to 20 to 25 atmospheres.

Each shutter is operated independently of the others, by means of the three-way cock in the engine-house, mentioned above. One shutter may be up, another down, and the others in any intermediate positions. When there is but little water in the weir, the shutters can be raised together from the accumulator in less than thirty seconds, but when the fall over the weir is 3 feet or more, the pump must be kept at work to assist the accumulator. In the latter case, the seven shutters can be raised in ten to fifteen minutes. It requires about two minutes to lower a shutter. This operation must not be performed too rapidly, in order to avoid scour. The maneuvering is facilitated by placing "butterfly-valves" in the shutters, which open when the shutter is down, so as only to present their thin edge to the pressure of the water.

When M. Girard first proposed his system, the objections were raised that his plan involved complicated machinery; that the jacks might become filled with sand;

that the water in the jacks or in their feed-pipes might freeze, etc. The trial of this system in the Ile Brulée dam has, however, been very successful. Freezing has been prevented by placing the jacks below water. During a severe winter, when the temperature fell to 13° Fahrenheit below zero, the only damage done was to some pipes in the engine-house, which had not been properly protected.

Owing to the size of the shutters and their being placed very closely together, the leakage through this dam is much less than what is usual with other systems of movable dams.

Desfontaines Drum-dam.—In 1846, M. Desfontaines invented and experimented with a new system of movable wickets which was introduced for the first time in the dam of Damery on the Marne and in 1861 on the Courcelles Dam. From 1861 to 1867 this form of wicket was placed on the overfall-weirs of nine of the dams on the Marne.

The arrangement of this invention is shown in Fig. 60. The dam is composed of sections about 5 feet long, which are placed on top of a masonry weir. Each of these

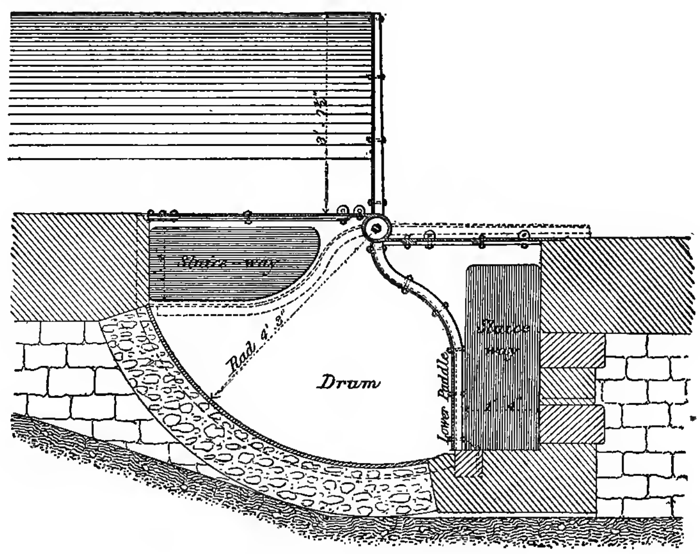

FIG. 60.—DESFONTAINES DRUM-DAM.

sections revolves on a horizontal axle placed at its centre. Its upper part, which acts like a shutter, is called the wicket proper; the lower portion, which serves to raise or lower the wicket, is called the counter-wicket. For convenience it is somewhat curved. The counter-wicket is placed in a chamber on top of the weir, which forms a quarter of a horizontal cylinder. In the first dams built according to this system the chamber was simply formed of sheet iron, but it was afterwards found advisable to surround it with masonry. The chamber was called "a drum," and from this the new device was called a drum-dam.

The counter-wicket is moved by admitting the water from the upper pool on its up-stream or down-stream side. In the former case it raises the dam; in the latter it lowers it. At the same time when the water from the upper pool is admitted to one side of the counter-wicket a connection is made on the other side with the lower pool, as explained hereafter.

The different sections composing the dam are placed side by side, so that their axles

shall be in the same horizontal line. They are separated in the drum by cast-iron diaphragms having on both sides projecting ribs corresponding to the extreme positions of the counter-wicket, and two openings or sluiceways placed as shown in Fig. 60. When the water from the upper pool is admitted at the end of the drum to the up-stream sluiceway

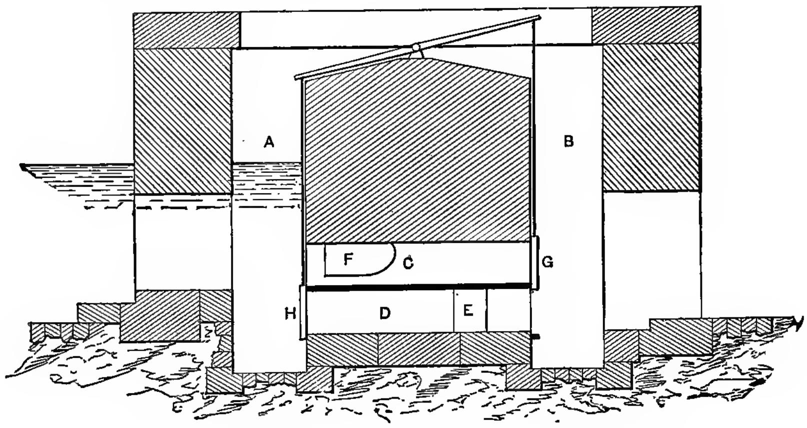

FIG. 61.—ABUTMENT OF DESFONTAINES DRUM-DAM.

of the first diaphragm, it presses the first counter-wicket down until it bears against a sill at the bottom and then flows from chamber to chamber, turning all the counter-wickets in succession, thus raising the dam. While this occurs, the sluiceways on the down-stream side of the counter-wickets are connected with the lower pool to avoid any counter-pressure by leakage between the different sections of the dam. In order to reduce this leakage as possible strips of rubber are attached to the bottom and sides of each counter-wicket to make tight joints where it bears against the sill and the projecting ribs of the diaphragms mentioned above.

The flow to and from either side of the counter-wickets is controlled at a pier or abutment built at the end of the dam, as shown in Fig. 61. Two wells are constructed in the pier or abutment, one, *A*, being connected with the upper pool and the other, *B*, with the lower. A rectangular gallery connects the two wells. It is divided by a horizontal metal plate into an upper and lower passage, *C* and *D*. The former is connected by a suitable channel *F* with the upstream side of the counter-wickets and the latter is connected by another channel *E* with their down-stream side. A sluice-gate of such size that it covers the upper or lower passage between the wells is placed at each end of the gallery connecting the wells. These two sluice-gates, *H* and *G*, are attached to a balance-beam in such a manner that when one gate covers the lower passage the other gate closes the upper one. By this simple arrangement all the wickets can be raised or lowered. In ordinary cases the lowering of the whole drum-dam would reduce the level of the upper pool too much. To avoid this difficulty the wickets of the early Desfontaines dams were provided with props somewhat like those of the Chanoine shutters. By suitable contrivances these props could be stopped at certain points when the dam was being lowered, or permitted to slide down completely. In this manner the distance to which the crest of the drum-weir was lowered could be regulated. It was found out, later on, that these props were not needed when the motion of the wickets was controlled

in the pier or abutment in the manner shown in Fig. 61. By setting the gates *H* and *G* so as to cover only partly the passages *D* and *C* a counter-pressure is produced which prevents the whole dam from being lowered. The wickets at the pier are lowered and those more remote remain up. By varying the position of the gates the number of wickets that are lowered can be controlled.

The largest Desfontaines Dam was constructed in France in 1867, across the overflow-weir of a dam on the Marne at Joinville. It consists of forty-two wickets, each being $3\frac{1}{2}$ feet high and $4\frac{3}{8}$ feet wide. This weir can be opened or closed by one man in three or four minutes. Larger drum-dams of this kind were constructed in 1883–86 across the timber-passes of four weirs on the Main, in Germany.* In this case each of the passes, which was $39\frac{1}{8}$ feet wide, was closed by a single wicket, retaining a head of 5.58 feet above the sill of the dam. Another large drum-dam has been placed across the navigable pass of the Charlottenburg Dam, on the river Spree.* It is formed of one wicket $32\frac{3}{4}$ feet wide, which retains $9\frac{1}{8}$ feet of water, the difference in level between the upper and lower pool being ordinarily 4 feet.

The drum-dam is undoubtedly the most perfect type of movable dam, as it gives perfect control over the wickets, and enables them to be raised even against a rapid current; but it is also the most expensive type, and its use is practically limited to overflow weirs, on account of the difficulty of constructing and maintaining the drum in a deep channel. The cost of the drum-dam at Joinville was about $135 per lineal foot. In the Charlottenburg navigable pass, the cost of the drum-dam reached $725 per lineal foot.

Cuvinot Drum-dam.—An improvement on Desfontaines' Drum-dam has been invented by M. Cuvinot.† The objects aimed at in the new design are:

1st. To make the wickets independent of each other.

2d. To arrange them so as to be stable in any position.

3d. To reduce the depth of the counter-wickets, and, consequently, the size of the drum.

4th. To diminish the leakage through the drum.

Fig. 62 shows the manner in which M. Cuvinot proposed to accomplish these objects. A semi-cylindrical and two rectangular conduits are constructed on top of the masonry weir for its whole length. The first-mentioned conduit forms the drum in which the counter-wickets are placed; the other two are the inlet and outlet conduits, the former being placed on the up-stream side of the drum and connected with the upper pool, while the latter is built on the down-stream side and connected to the lower pool. The drum is divided into compartments, corresponding to the length of the wicket, by tight diaphragms, which support the axles of the wickets and counter-wickets. Each of these compartments is connected with the inlet-conduit by a hole that always remains open. Another opening, which can be controlled by a valve, connects it with the outlet-conduit. The counter-wicket has a short arm projecting out of the drum and provided at its extremity with a friction-roller, which bears against the down-stream side of the

* Rivers and Canals, by L. F. Vernon-Harcourt, M.A., Vol. I., p. 143.

† Cours de Navigation Intérierure, by H. Lagrené, Vol. III., p. 319.

wicket. The water from the upper pool fills every compartment of the dam, as the counter-wickets do not make tight joints at their bottom and sides. If the outlet-valve of any compartment be closed, the water in the compartment remains in equilibrium, and the pressure of the upper pool against the wicket turns the dam down. If the valve be opened, the pressure on the down-stream side of the counter wicket is at once reduced, and the counter-wicket revolves and raises the wicket by

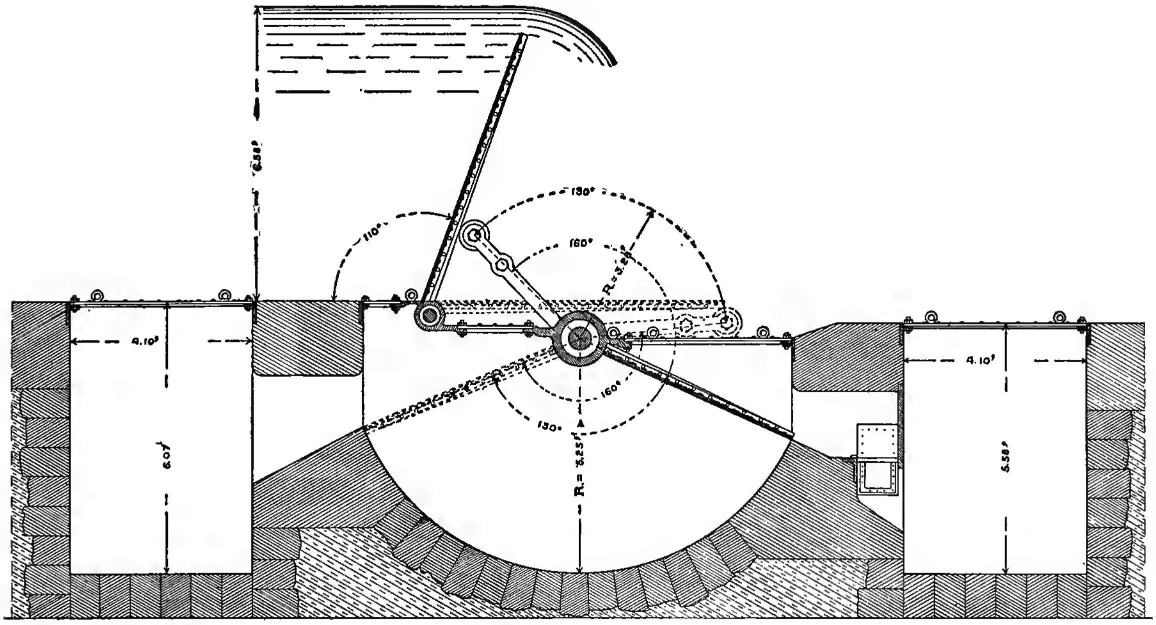

FIG. 62.—CUVINOT DRUM-DAM.

means of its projecting arm. The extent to which the valve is opened determines the difference in pressure against the two faces of the counter-wicket and regulates the height to which the wicket is raised. By arranging the outlet valves of the different compartments so as to be worked from either end of the weir, the dam-tender can control the position of every wicket. Fig. 62 shows the two extreme positions which the wickets and counter-wickets can assume.

Chittenden Drum-dam.—An improved type of drum-dam, invented by Capt. H. M. Chittenden, Corps of Engineers, U. S. A., is to be used in the improvement of the Osage River, in a dam having a maximum difference of 16 feet between the upper and lower pools.* Fig. 63 shows the manner in which this dam is to be constructed. It rests entirely on a pile foundation, which is covered by a water-tight floor of 4-inch plank, except at the apron. The second row of piles from the up-stream face consists of a triple thickness of sheet-piling. A block of concrete *HIJK* forms the fixed part of the dam. The movable part consist of a box-gate having the sector of $67\frac{1}{2}°$ of a circle as its cross-section and revolving around the axle *A*. The interior framework of the gate is made of iron, but the outside consists of wood. The ends of the gate

* Paper on "Modified Drum-weir," by H. M. Chittenden, in Journal of the Association of Engineering Societies, June, 1896.

(or of different sections forming it) are closed and made air-tight from C down for about one-third the height. The upper face, AB, and the cylindrical face for about two-thirds the distance from C to B, are also made air-tight. The lower face is water-tight. When down the gate falls into the chamber AZQ and rests against the step Z. This chamber is constructed in the following manner. Iron frames $DEFG$, which support the axle of the gate and are anchored into the concrete, are placed 5 feet apart. On the opposite side of the chamber iron frames $LMNOPD$ are put

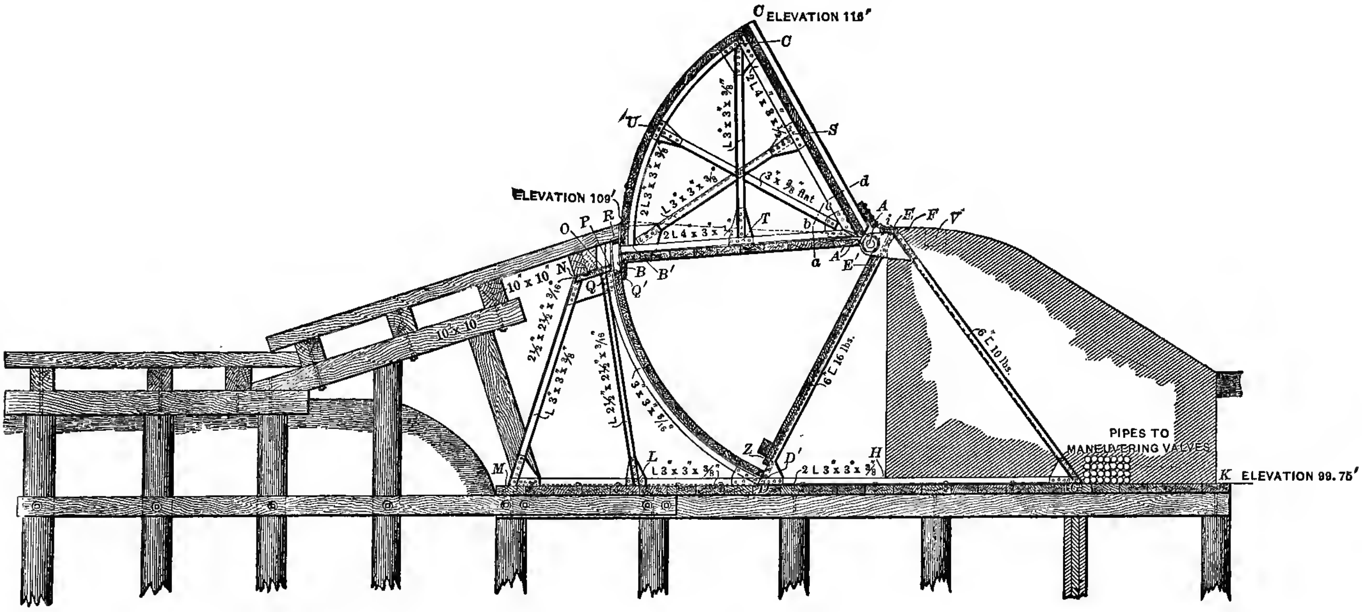

FIG. 63.—CHITTENDEN DRUM-DAM.

up every $2\frac{1}{2}$ feet. Water-tight wooden partitions $D'Q$ and $E'Z$, supported by the iron frames, form the two sides of the chamber, which has a cross-section corresponding to that of the gate. The frames, $LMNOPD$, carry also the upper part of the timber apron.

The triangular space DEH forms a longitudinal culvert for conveying water to or from the chamber AZQ. The inlet to the chamber consists of a narrow opening at Z extending the whole length of the gate so as to admit the water uniformly under the face AB of the gate and having an area slightly in excess of that of DEH. The flow into the culvert DEH and thus into the chamber AZQ is regulated by suitable gates placed in the piers which separate the different sections of which this drum-gate is composed. When the water from the upper pool is let into the chamber AZQ there is generally enough head to raise the gate to its normal position when up. Should there not be sufficient head to raise the gate, its buoyancy is increased by admitting air into the gate from an air-pump on shore and expelling some of the water until it rises to its normal position. As the gate rises to its normal position it can be stopped by means of the inlet-valve or automatically by letting the water escape at openings at Q. Seven of these openings, having together an area of 10 square feet, are provided for each section of the drum. The inlet

culvert has an opening of 12 square feet, but as there is always some leakage, the openings at *Q*, when entirely uncovered by the drum, can discharge all the water conveyed by the inlet-culvert. The drum remains, therefore, stationary when it has reached its highest position.

In devising this gate Captain Chittenden's object was to overcome the defects inherent in the bear-trap gate (page 192). Compared with the latter, the Chittenden gate has the following advantages:

1st. It has but one axis of motion.

2d. It has no angles in which drift can lodge.

3d. It can easily be built in sections.

4th. It contains no sliding surfaces causing friction.

CHAPTER III.

DAMS WITH BEAR-TRAP GATES.

History of the Gate.*—In 1818, Josiah White, a Philadelphia merchant, and Erskine Hazard, the managers of the Lehigh Navigation Company, commenced the improvement of the Lehigh River, to obtain slack-water navigation for transporting anthracite coal to market. At first the improvement consisted only in the construction of wing-dams, but this did not furnish the minimum water-way required by the charter of the company, viz., a width of 25 feet with a depth of 18 inches. White and Hazard had, therefore, to seek other means of increasing the depth of the water. They determined to accomplish this by producing artificial freshets by means of some kind of movable gate which was to be placed across the river. White spent several weeks in trying to construct a satisfactory device for this purpose, and finally produced the well-known bear-trap gate. He built a small gate of this character in the Mauch Chunk Creek, which excited much attention during its construction. The workmen employed tried to rid themselves of curious people, who wished to find out what Mr. White was constructing, by telling them that it was a "bear-trap," a name the gate has borne ever since. In 1819 twelve dams having gates of this kind were placed in the Lehigh River, and proved to be perfectly successful. Later on this gate was used on the logging streams of Pennsylvania and Canada.

Some bear-trap gates were built also in France, and have remained in use in smaller streams. A larger gate of this kind, constructed for the river Marne (Fig. 65), not properly proportioned, was a failure. As the results were widely published without the causes of the failure being carefully examined, a prejudice against this style of gate became deeply rooted among engineers for many years. The gate fell almost into disuse, excepting on logging streams in Pennsylvania, where it has remained in use to the present time.

One of the first engineers to revive the interest in the bear-trap gate was the late Ashbel Welsh, President of the American Society of Civil Engineers, who, in his annual address delivered to that Society in 1882, stated:

"The bear-trap locks (White's on the Lehigh) have given the hint for several devices since used, and are well worthy of an examination.

"It is well-worth inquiry whether these bear-trap gates would not be the best possible, and possibly the cheapest, for letting the water rapidly out of a reservoir for scouring purposes. A full stream could be set running in a few seconds, and the flow could be regulated with perfect ease and stopped at any moment."

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* Paper on "Movable Dams, Sluice- and Lock-gates of the Bear-trap Type," by Archibald O. Powell, read before the Civil Engineers' Society of St. Paul, and published in the Journal of the Engineering Societies for June, 1896.

Soon afterwards, in 1883–84, a board of United States Engineer officers recommended the adoption of two bear-trap gates in the passes of the Beattyville Dam on the Kentucky River. These gates, each of which is 60 feet long, were built in 1886. They proved to be satisfactory, although they might have been better proportioned. In 1888 a bear-trap gate 52 feet wide was placed in the drift-pass of the Davis Island Dam on the Ohio River (Fig. 64).* The construction of this gate caused the United States Engineers to make a theoretical study of the bear-trap gate. This work was assigned, in 1892, to Captain Chittenden, U. S. A., and Mr. Archibald O. Powell, United States Assistant Engineer.†

Description of the Gate.—As originally constructed the gate consists of two rectangular leaves of a length equal to the width of the opening in which they are placed (Fig. 64). Each of the leaves has at the bottom an axle or hinges which enables it to revolve. When the gate is down the up-stream leaf overlaps the down-stream leaf. The gate is raised by the pressure of the water from the upper pool, which is conveyed in a channel controlled

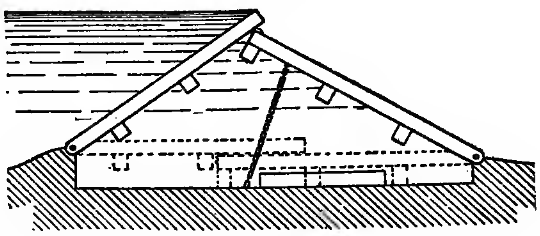


FIG. 64.—BEAR-TRAP GATE, DAVIS ISLAND DAM.

by a sluice-gate to a chamber constructed under the gate. A second channel, also provided with a gate or stop-cock, connects this chamber with the lower pool. When the connection with the upper pool is opened while that with the lower pool is closed, water from the upper pool fills the chamber under the gate. This causes the down-stream leaf to rise, first by flotation and then by the impulse from the flow of the water. In rising, the lower leaf raises the upper leaf by its edge sliding under it, the friction being reduced by rollers. The height to which the gate rises is limited either by stay-chains attached to the lower leaf or by a piece of wood nailed on the under side of the upper leaf. In lowering the gates, the operation is reversed, the connection with the upper pool being closed while that with the lower pool is opened. By regulating the extent to which the two valves controlling the inlet and outlet of the chamber under the gate are opened the gate may be made to assume any intermediate position between the two extremes mentioned above.

Although very simple in principle, the old "bear-trap gate" contains a number of objectionable features. The overlap of the upper over the lower leaf necessitates lifting a considerable amount of water when the gate is to be raised. The head obtained is only about one-third of the total length of the gate. The friction between the two leaves, even when

* Figs. 64 to 68 (except 65) are taken from the paper of Mr. Powell mentioned on page 191.

† The results of these investigations were given by these engineers in papers read before the Civil Engineers' Society of St. Paul, which, with several others on "American Hydraulic Gates, Weirs, and Movable Dams," written by other engineers, have been published in the Journal of the Association of Engineering Societies for June, 1896.

reduced by rollers, makes it difficult to operate the gate smoothly. As the gate is not adapted to being made in sections, its width has to be equal to the opening which it is to close. With a wide gate, however, one side is apt to go up faster than the other, causing twisting strains in the leaves.* The sudden stopping of the gate when it has reached its proper height causes great strains and is apt to damage the gate. Any driftwood or stone that may lodge between the leaves makes the lowering of the gate impossible. These defects have fortunately been overcome by recent improvements and modifications in the construction of this style of gate, and it seems now very probable that the bear-trap gate in some improved form will be used largely in the construction of movable dams and also for locks.

The three principal problems to be solved in connection with bear-trap dams are:

1st. To secure the power to start the gates when they are to be raised.

2d. To prevent warping and twisting during the raising and lowering.

3d. To construct the gate so that it can be used for passes of considerable width without having intermediate piers.

Some initial head is always required to start the gate when it is to be raised. Mechanical means for starting the raising were used in some of the early gates. In the Neuville Dam on the Marne, in France (Fig. 65), which has a sluice closed by bear-trap

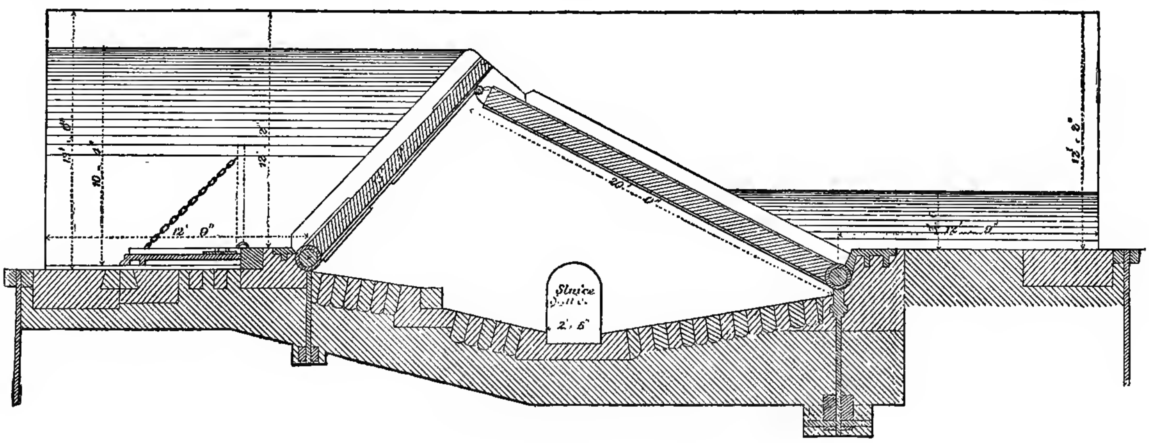


FIG. 65.—BEAR-TRAP DAM ON THE MARNE.

gates, the initial head for starting them is obtained by means of an auxiliary dam consisting of Thénard counter-shutters. An experiment is to be made to start the bear-trap gates of Dam No. 6 on the Ohio River by compressed air, but this involves an expensive plant.

The most favorable conditions exist when the required head for starting the gates can be obtained from some auxiliary reservoir. When this is impracticable or too costly, the "head" will generally have to be obtained by some auxiliary dam, as at Neuville, although some cheaper construction may be devised.

* In a bear-trap gate 120 feet long and about 9 feet high, built in Dam No. 1, of the Monongahela River, the warpage was so great in raising the gate that the end next to the filling culvert came up 5 feet in advance of the opposite end. In lowering, the end at the culvert was also about 5 feet in advance of the other end, so that the variation in the crest amounted to 10 feet. (Journal of the Association of Engineering Societies for June, 1896, p. 209.)

Uniformity in the movement of the gate can be insured, where the width of the pass is not too great, by admitting or drawing off the water at both ends. If the gate is made as tight as possible the water can be admitted slowly, which insures uniformity of motion. Long gates should be constructed in such a manner that the water can be admitted uniformly under the gate not only at the ends, but at some intermediate points.

Mr. Dubois tried, but without success, to insure uniformity of motion by attaching a shaft having pinions at regular intervals, to the floor of the chamber. The pinions worked racks fastened to spuds which were hinged to the upper end of the lower leaf. By this arrangement the motion of the leaves revolved the shaft and moved the spuds simultaneously. A somewhat similar arrangement of racks and pinions has been experimented with at the Davis Island Dam on the Ohio.* The amount of head required for this purpose depends evidently upon the correctness of the design of the gate.

As regards the width of pass which can be closed with one bear-trap gate, the question has not yet been determined. Maj. William L. Marshall, U. S. A., and other engineers who have been studying this problem, have devised means which will probably permit bear-trap gates of considerable length to be constructed. Mr. William Martin, U. S. Ass't Engineer, who experimented with the bear-trap dam at Davis Island, states:† "I am firmly of the opinion that the old-style bear-trap, properly proportioned and built in a substantial manner, reducing leakage to a minimum, is capable of successful use in spans up to lengths from 200 to 300 feet." ‡

DuBois Gate (Fig. 66).—In 1862 Mr. DuBois, of Williamsport, Pa., patented a modification of a bear-trap gate. His invention consisted in joining the two leaves by a hinge placed at the apex. The lower leaf was also hinged to the foundation, but the foot of the upper leaf was not attached and was to slide on the foundation during the lowering of the

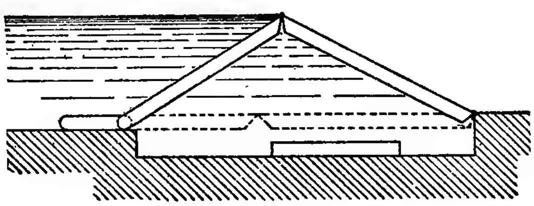


FIG. 66.—DUBOIS GATE.

gate. By this arrangement the friction existing between the leaves in the old-style gate is simply transferred to the bottom of the upper leaf. As this leaf has to slide back under the full head during the lowering the advantage gained by placing the hinge at the apex is neutralized. The DuBois gate is, therefore, not much of an improvement and can only be considered as a modification of bear-trap gates.

Carro Gate (Fig. 67).—A French engineer, M. Carro, invented in 1870 a gate which is very similar to that of DuBois. The leaves are hinged at the apex, but both slide on the foundation during the lowering of the gate, the motion being limited by links fastened to the lower leaf and to the foundation. To prevent drift and sediment from interfering with the

* Journal of the Association of Engineering Societies, Vol. XVI., p. 209. † *Ibid.*

‡ A bear-trap gate 160 feet wide has been constructed for the Chicago Drainage Canal.

sliding of the upper leaf, a special leaf, hinged to the foundation and bearing against the upper leaf, is placed as a screen.

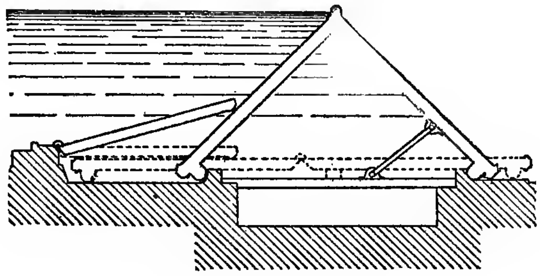


FIG. 67.—CARRO GATE.

Girard's Gate (Fig. 68).—In 1868 M. Girard took out a French patent for a gate of the bear-trap style, in which the sliding of the leaves was entirely done away with by placing one hinge at the apex and another near the middle of the lower leaf, which is fastened to a stay-

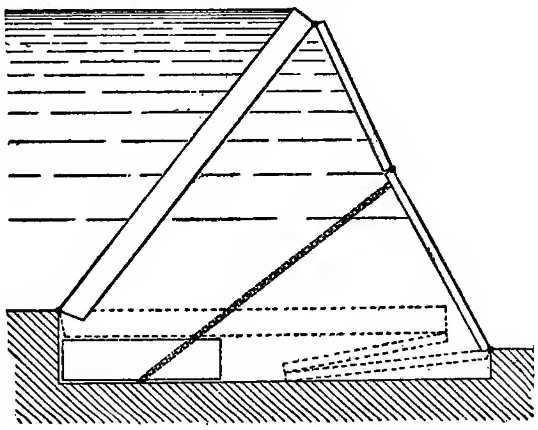
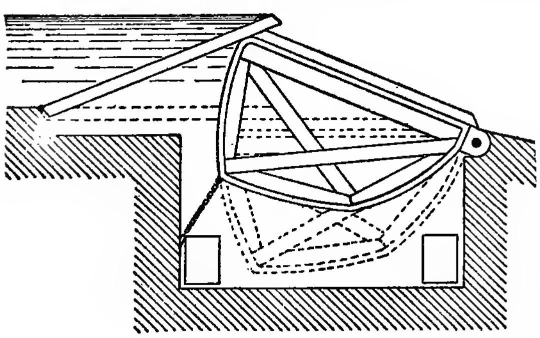


FIG. 68.—GIRARD GATE.

chain. By this arrangement the lower leaf is made to fold up when the gate is lowered. Although this invention constituted a decided advance in the construction of bear-trap gates, it does not appear to have been practically introduced or to have had its merits appreciated.

Brunot Gate.—The Hon. Felix R. Brunot, of Alleghany, Pa., patented in 1867 a sluice-



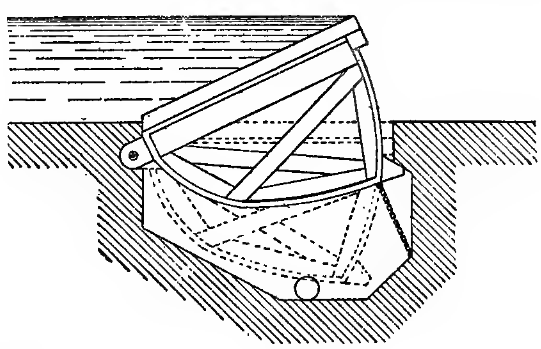


FIG. 69. BRUNOT GATE. FIG. 70.

gate for dams and locks which consisted of a bear-trap gate having its lower leaf moved by a pontoon (Fig. 69). In a later design the lower leaf was dispensed with, the pontoon being

placed directly under the upper leaf (Fig. 70). Two means of operating the gate were proposed. The first consisted in floating or sinking the pontoon in a conduit in a manner very similar to that proposed by M. Krantz (page 181). The second, which was probably the better plan, consisted in changing the buoyancy of the pontoon by admitting or pumping out water. A turbine-wheel, placed in a well in the abutment, and worked by the head of the upper pool, was to furnish the power for pumping.

Parker Gate (Fig. 71).—This gate, which was patented in 1887 by Thomas Parker, Menominee, Wis., is really the Girard gate turned around so that the upper instead of the lower leaf has a hinge near its middle. To prevent drift, etc., from interfering with the folding of the upper leaf, a special leaf, called an "idler," is introduced. It is hinged at the apex. Its bottom edge slides on the floor up-stream during the lowering of the gate, but as the pressure on both sides of the idler is the same, the friction caused by the sliding is insignifi-

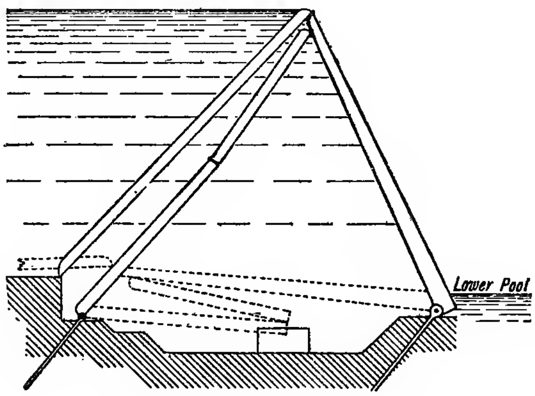


FIG. 71.—PARKER GATE.

cant. Grated openings are provided in the idler to permit the water to circulate freely around it. Most of the defects of the old style bear-trap gate are overcome in Mr. Parker's device. The length of the gate is reduced, the friction due to sliding is eliminated; the gate is obliged to rise uniformly; it is not stopped suddenly, and the idler prevents all trouble from drift. A number of these gates have already been constructed and have answered all the requirements. The largest one of these gates thus far constructed is the one built in 1892 for the Muscle Shoals Canal in Tennessee. It is 40 feet long and 8.5 feet high.

Mr. Parker designed, also, some of his gates to be used in a reversed position, with the hinge in the down-stream leaf.

The Lang Gate (Fig. 72), patented by Mr. Robert A. Lang, of Eau Claire, Wis., in 1890, was intended to be an improvement on the Parker gate. In Lang's gate the idler is made an essential part of the device and the upper part of the upper leaf is dispensed with, rods or chains taking its place. In this gate we have sliding friction again, viz., between the bottom of the idler and the upper leaf. Of course, it may be much reduced by using rollers. It is claimed that when the friction is at its maximum it is overcome by the weight of that part of the gate which is suspended in air. Engineers are still divided in opinion as to whether the Lang gate is really an improvement on the one devised by Parker. Further experience will have to settle this question.

A number of Lang gates, 20 to 80 feet long and 7 to 14 feet high, are now in use in the United States.

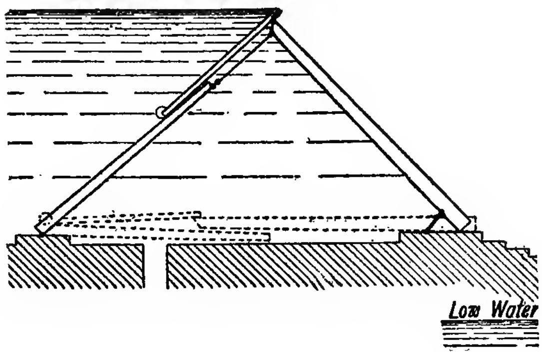


FIG. 72.—LANG GATE.

Marshall Gate.—Fig. 73 shows one form of a gate invented and patented in 1895 or '96 by Major William L. Marshall, Corps of Engineers, U. S. A. It differs in two essential

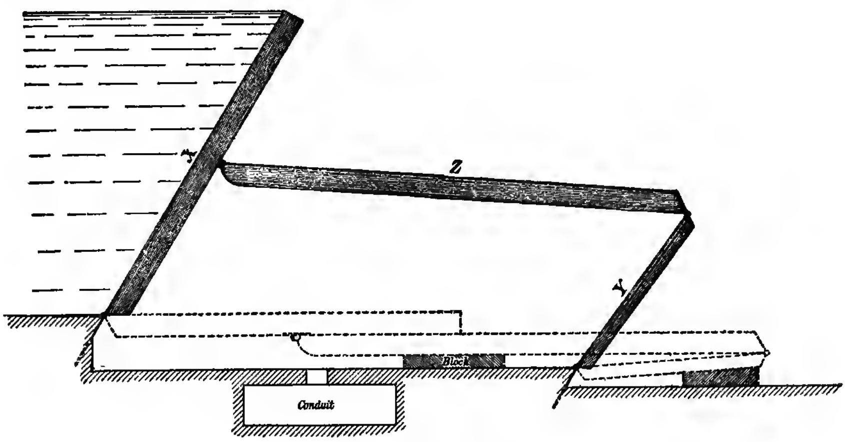


FIG. 73.—MARSHALL GATE.

points from the other types of bear-trap gates described in this chapter: 1st. The hinge joining the leaves is placed near the middle of one of the leaves instead of at its end; 2d. The hydraulic chamber under the leaves has a quadrangular instead of triangular cross-section, all of its angles being salient.

The arrangement of this gate makes it very flexible and does away with the necessity of restraining chains or stops.

Besides the gates mentioned above, other similar devices of little or no merit have been patented.* For a fuller discussion of this subject, including theoretical investigations, we must refer the reader to the series of papers published in the Journal of the Association of Engineering Societies for June, 1896. An article on bear-trap gates, written by Capt. H. M. Chittenden, U. S. A., was published in the Engineering News for February 7th, 1895.

* Wood, in 1871; Werner, in 1873; and Smith in 1875.

APPENDIX.

SPECIFICATIONS FOR THE NEW CROTON DAM.

(1) **Plans.**—The plans referred to in these specifications are twelve (12) in number, entitled "The Aqueduct Commission, Contract Drawings, New Croton Dam at Cornell Site," Sheets 1 to 12 inclusive, and signed by the Chief Engineer, and dated May 2d, 1892.

They show the location of the work, and its general character. During the progress of the work, such working plans will be furnished from time to time by the Engineer, as he may deem necessary.

(2) **Test-pits and Borings.**—Test-pits and borings have been made to ascertain the nature of the ground where the work is to be built; should the character, location and extent of the various materials be found to differ from what is indicated by the test-pits and borings, the Contractor shall have no claim on that account, and it is expressly understood that the Corporation of the City of New York does not warrant the indications of the tests to be correct.

(3) **General Description of the Work.**—The Dam is to be erected across the valley of Croton River, about 3¼ miles below the present Croton Dam, approximately at such point of the tract of land designated as Cornell Site as is indicated on sheet No. 3. The central part of the Dam is to be wholly of masonry built on the solid rock, as shown on the plans, or as may be hereafter ordered by the Engineer. On the right bank of the river a deep gate-chamber and beyond it a long spillway or overflow, with a channel connected therewith following the contour of the side-hill, all of masonry, are to be built. The water flowing over the spillway is to be conducted to the bed of the river below the Dam by means of a channel excavated in the side-hill. On the left bank of the river and in continuation of the central part of the Dam, an embankment containing a central masonry wall, built on the solid rock, is to be erected; a gate-chamber is to establish a communication between the old Aqueduct and the proposed Reservoir. Heavy paving or sodding is to protect the surfaces of the embankments.

A channel is to be excavated for the purpose of diverting the waters of the river before work can be begun near its present bed, and roads must be built to turn and maintain the public traffic, which must not be interrupted by the operations of construction.

All earth, rock and timber work, all iron work, all masonry work and all other work of a permanent or temporary character, necessary to complete the Dam, are described in this agreement.

During the progress of the work below the level of the river, it will flow in the temporary channel provided for it. As the Dam rises to a higher level, openings shall be left in the masonry to accommodate the flow of the river; but, inasmuch as in heavy freshets the said openings would not be sufficient, the water may rise behind the Dam, and, as it might overtop it, a part of the Dam masonry nearest to the river-bed is to be always left at a lower level, at a point where the fall of the water cannot cause any permanent injury.

All work, during its progress, and on its completion, must conform truly to the lines and levels to be given hereafter and determined by the Engineer, and must be built in accordance with the plans and directions which shall be given by him from time to time, subject to such modifications and additions as said Engineer shall deem necessary during the prosecution of the work; and in no case will any work which may be performed, or any materials furnished in excess of the requirements of this contract or of the plans, or of the specifications, be estimated and paid for unless such excess shall have been ordered by the Engineer, as herein set forth.

The Contractor is to furnish all materials (except such as may be obtained from the excavations), and all tools, implements, machinery, and labor (necessary or convenient for doing all the work herein contracted for, with safety to life and property in accordance with this contract, and within the time specified herein) required to construct and put in complete working order the work herein specified, and is to perform and construct all the work covered by this agreement; the whole to be done in conformity with the plans and these specifications; and all parts to be done to the satisfaction of the said Aqueduct Commissioners.

(4) **Methods and Appliances.**—The Contractor is to use such methods and appliances for the performance of all the operations connected with the work embraced under this contract as will secure a satisfactory quality of work and a rate of progress which, in the opinion of the Engineer, will secure the completion of the work within the time herein specified. If, at any time before the commencement, or during the progress of the work, such methods or appliances appear to the Engineer to be inefficient or inappropriate for securing the quality of the work required or the said rate of progress, he may order the Contractor to increase their efficiency or to improve their character, and the Contractor must conform to such order; but the failure of the Engineer to demand such increase of efficiency or improvement shall not relieve the Contractor from his obligation to secure the quality of work and the rate of progress established in these specifications.

PROTECTIVE WORK.

(5) **Highways.**—As the present highway is to be interrupted at the beginning of the work, new highways, bridges and culverts must be built first, to turn and maintain safely the public traffic, also all fences and other appurtenances necessary for public safety.

(6) **Diverting Channel and Temporary Dams.**—Before operations can be begun about the bed of the river, an artificial channel must be provided for the river. It is to be excavated in the side-hill, and two or more temporary dams are to be built in connection with it for the purpose of excluding the water from the portions of the valley into which excavations are to be made. The size and disposition of these structures are indicated on the plans.

(7) **Responsibility of the Contractor.**—The Contractor shall do all other work needed to protect his work from water; he shall erect all temporary dams, coffer-dams, sheet-piling and other devices, take care of the river, and shall be responsible for all damage that may be caused by the action of water, whether from negligence or any other cause. Such damage is to be repaired, and the work must be restored and maintained at his cost.

(8) All earth and rock excavation, masonry, timber and other work, temporary or permanent, for the purpose of protecting the work from the river, provided that they are ordered or approved by the Engineer, are to be paid for at the prices stipulated in this contract. All work of this character is to be removed by the Contractor at his own expense, if so ordered by the Engineer.

The responsibility of the Contractor as to damage caused by the inefficiency of the protective work shall cease, however, if such damage is caused by the river at a time when the flow of the river attains such volume as will cause it to rise to a height of more than eighty-one inches above the stone crest of the present Croton Dam, such height being the greatest recorded by the City authorities.

Such damage as may be caused under the circumstances just stated shall be repaired by the Contractor as soon as practicable, under the direction of the Engineer, who shall appraise the cost of such work of repairs, and the amount of the same shall be paid to the Contractor on the certificate of the Engineer that the work has been completed to his satisfaction; and, after such certificate shall have been issued, the Contractor shall again become responsible for all damage that may be caused by the action of the water, in the same manner as is specified in clauses 7 and 8.

If such appraisal of the Engineer is not satisfactory to the Contractor, the said Contractor shall so state in writing to the Aqueduct Commissioners, and, thereupon, a Board of Arbitration, composed, 1st, of the Chief Engineer, or of such other person that the Aqueduct Commissioners may designate; 2d, of a person selected by the Contractor; 3d, of another person to be designated by the other two, shall proceed to appraise the cost of such damage, and their decision shall be final and binding on both parties, provided it is the unanimous decision of the three members of the said Board; but if the said decision is not unanimous, the appraisal of the Chief Engineer shall stand and become final and binding to both parties. And, on the certificate of the Aqueduct Commissioners that the said appraisal has been made in accordance with the stipulations of this agreement, the amount of said appraisal shall be paid to the Contractor. And the said appraisal, whether made by the Chief Engineer or by the said Board of Arbitration, shall include only the cost of the actual work done to repair the damage, and shall not include any alleged loss of profit or other loss due to the delay caused by such repairs, but an extension of time shall be granted to the Contractor

for the performance of his contract, equivalent, in the opinion of the Engineer, to the loss of time due to the interruption of the operations of construction on account of the said work of repairs.

(9) **Pumping.**—The Contractor is to do all the draining and pumping which shall be necessary for keeping the work free from water, and if at any time the Engineer is of opinion that, in order to maintain the slopes and sides of the excavations in proper order, it is necessary to remove the water from the ground outside of the limits of the excavations, the Contractor shall, at his request, sink the necessary pipes or wells to intercept the water, and place, maintain and work such pumping or other exhausting apparatus as shall be sufficient to properly maintain the said slopes and sides.

The cost of furnishing the necessary appliances and machinery, of working them, and of doing all the work connected with draining and pumping operations, is to be included in the prices bid for the various kinds of work which the draining and pumping operations are intended to protect.

SOIL.

(10) The soil is to be removed from the grounds where the Dam, embankments and other works are to stand and the excavations to take place, and wherever directed by the Engineer, and shall be deposited as directed or approved by him.

It shall be estimated as earth excavation clause O, item (*c.*),* and if of proper quality, may be used afterward by the Contractor.

The slopes of the embankments and such other places as may be designated, shall be covered with soil which the Contractor may take from his spoil-banks or elsewhere; it must be of good quality and, after being rolled or otherwise compacted, it shall be measured in embankment and paid for as stipulated in clause O, item (*a.*).

The thickness of the soil shall be six inches or more as may be ordered.

SODDING.

(11) The embankments of the Dam and such other surfaces as may be designated by the Engineer, are to be sodded.

All the surfaces to be sodded are to be carefully graded, so as to make a true and even bearing for the sods to rest on.

The sods to be of good quality of earth covered with heavy grass, sound and healthy, and not less than one foot square, and generally of a uniform thickness of three inches (which sizes may be altered by the Engineer during the progress of the work); to be cut with a bevel on all sides, so that when laid they will lap at the edges; to be properly set so as to have a full bearing on their whole lower surface; to be padded down firm with a spade or wooden bat made suitable for the purpose; each sod is to be pinned with one wooden pin to each sod, not less than fifteen inches long, so as to be secured to the ground beneath it, and to be so laid that the upper surface shall conform to the true slope of the bank or ground, and to the lines given

* Clause O of the contract contains the prices for the different items of work.

by the Engineer. No lean, poor or broken sods will be allowed in the work, but on the outside edges of the bank, sods may be cut to such size and shape as will make a proper finish to the same.

(12) The sodding that shall have been laid shall be well and carefully sprinkled with water as often as the Engineer shall deem necessary for the benefit of the work during its progress.

EARTH EXCAVATION.

(13) **Earth Excavation.**—The earth excavation is to be very extensive and, at many points, of great depths. It is to be made for the construction of highways, for the uncovering of the rock, for the preparation of the spaces into which masonry and other work is to be built, for trenches, and for any other work which the Engineer may order.

(14) **Measurements.**—All earth excavation is to be measured according to the lines and slopes established from time to time by the Engineer. The plans indicate in a general way the slopes of the excavations, but they will be modified during construction in accordance with the character of the materials encountered. When they are so modified, the Contractor shall conform to the modified lines given from time to time without extra compensation on account of such modification.

(15) **Depths.**—The depth at which the sloping excavations are to be abandoned and the vertical trenches are to be begun for the centre walls, will depend also on the character of the materials encountered, and cannot be fixed in advance.

(16) **Timbering.**—The timbering of all trenches must be done with great care and shall be conducted by skilful mechanics.

(17) **Disposal.**—The materials excavated must be deposited in such a manner, at such places, and at such distances from the excavations as shall be directed or approved by the Engineer.

All work done under this head is to be measured in excavation.

(18) **Prices.**—The prices herein stipulated for earth excavation (clause O, items (*c.*) and (*cc.*)) are to include the work of clearing and grubbing the grounds of all trees, stumps, bushes and roots, and the burning or otherwise disposing of the same; of sheeting and bracing, and of supporting and maintaining all trenches and pits during and after excavation; of all pumping, ditching and draining, and of disposing of the excavated materials.

(19) **What is Earth Excavation.**—All excavation of earth, hard-pan and other materials, including boulders not exceeding one cubic yard each, shall be classified and estimated as earth excavation and paid for at the prices herein stipulated (Clause O, items (*c.*) and (*c c.*)).

(20) **No Extra Haul.**—No extra haul shall be paid for materials excavated under this head.

(21) **Excavation for Highways.**—For the work of constructing highways all the materials shall be measured in excavation, including such as may be borrowed outside of the work, and the price bid (clause O, item (*c.*)) shall include the cost of disposing

of the excavated materials in forming the roadbed and in making the fills and embankments connected with the highways and with their appurtenances.

(22) **Two Prices.**—Two prices are to be paid for earth excavation. One price (clause O, item (*c.*)) for all earth excavation other than that made in vertical trenches. Another price (clause O, item (*c c.*)) is to be paid for all excavations made in vertical trenches for the purpose of building therein the centre wall of the embankments and for the purpose of placing sheet-piling for the protection of river work.

REFILLING AND EMBANKMENTS.

(23) The work to be done under this head consists of all the earth and broken rock work necessary for refilling the excavations and for making embankments (except for construction of highways as hereinbefore specified, clause (21)).

(24) **Measurement.**—All the materials used for refilling and embankments are to be measured according to the dimensions of the spaces which are to be refilled and of the embankments in place.

(25) **Where Taken.**—The materials necessary for refilling and embankments are to be taken from the dumps formed during the process of excavation or from approved borrow-pits.

(26) **Extra Haul.**—Whenever the materials used for refilling and embankments are to be hauled a distance greater than three thousand feet measured on a straight line from the borrow-pit to the nearest point on the centre-line of the base of the Dam, an additional price equal to three per cent of the price stipulated for refilling and embankment (clause O, item (*d.*), is to be paid to the Contractor for each yard for each one hundred feet that the said yard is hauled farther than the said three thousand feet.

(27) **How Made.**—The embankments for the main and temporary Dams shall start from a well-prepared base, stepped on sloping ground; all embankments and all refilling shall be carried up in horizontal layers not exceeding six inches in thickness; every layer to be carefully rolled with a heavy grooved roller, and to be well watered. The earth to be well rammed with heavy rammers at such points as cannot be reached by the roller. Special care shall be required in ramming the earth close to the sheet-piling and to the masonry, which shall always be kept at least two feet higher than the adjoining embankment, unless otherwise permitted. The embankments of the Dam shall be kept at an uniform height on both sides of the masonry during construction, unless otherwise permitted.

(28) Ample means shall be provided for watering the banks, and any portion of the embankment to which a layer is being applied shall be so wet, when required, that water will stand on the surface. The Contractor shall furnish at his own cost the necessary steam or other power for forcing the water upon the bank, if the Engineer find that other means of transportation and distribution of the water are not sufficient.

(29) **Extra Thickness of Embankments.**—The embankments of the Dam or any slopes that may be so ordered shall be formed with an extra width of twelve inches; this surplus quantity of earth shall be afterwards removed and estimated as excavation (item (*c.*)), and the surface left shall be dressed smoothly to receive the broken stones supporting the paving or the soil.

(30) **Quality.**—The earth used for the embankments shall be free from perishable material of all kinds and from stones larger than three inches in diameter, and it shall be of a quality approved by the Engineer.

(31) **Selected Materials.**—The Engineer shall decide upon the quality and character of the earth to be used at various places, and it must be selected and placed in accordance with his orders; the most compact material must be used on the up-stream side of the centre wall of the Dam embankments and for the refilling of the wall trenches; more porous material must be used for the embankments on the down-stream side.

(32) **Mixing.**—When the Engineer finds it necessary to mix the materials to be used for making embankments and for refilling, separate loads of the various materials designated by the Engineer, and in proportions to be determined by him, shall be deposited on the embankments at proper intervals; they will then be thrown with shovels or otherwise in such a manner as to effect a thorough mixture.

(33) **Borrow-pits.**—The borrow-pits must be acceptable to the Engineer, but the City shall not pay for the removal of boulders, trees, stumps and other things which are not acceptable for refilling and embankments.

(34) **Price.**—The price herein stipulated for refilling and embankments (clause O, item (*d.*)) is to include the cost of excavating and taking the materials used therefor from the dumps or from the borrow-pits, of supporting, draining and maintaining the excavations, of selecting, mixing and transporting the materials, of rolling and watering, and of doing all work necessary for placing the same as hereinbefore specified.

(35) **Broken Rock for Refilling and Embankments.**—All excavated rock taken from the excavation or from the places where it has been deposited, and used in the same manner as herein specified for refilling and making embankment, shall be classified under that head, and shall be paid for at the price stipulated (clause O, item (*d.*)).

ROCK EXCAVATION.

Rock excavation is to take place for the channel through which the river is to be diverted, for the spillway channel, for the foundations of the Dam, of the Gate House and of the centre wall, and wherever the Engineer may order it.

(36) **Rock Excavation Defined.**—Rock excavation is to include the excavation of all solid rock which cannot be removed by picking, and of boulders of one cubic yard or more in size and the removal of masonry.

(37) **Measurements.**—Rock is to be measured in excavation to the lines determined by the Engineer.

(38) **Stepping.**—In the wall and pipe trenches and in the excavations for the Dam, Gate House, overflow, spillway channel and other structures, the rock is to be shaped roughly in steps or other form that may be ordered by the Engineer.

(39) **Price.**—The price bid for rock excavation is to include the cost of supporting and maintaining the excavations, of pumping and draining, of disposing of the excavated materials as approved by the Engineer, and all other incidental expenses.

(40) **Explosives.**—All rock excavation in the wall trenches and at any other place designated by the Engineer is to be made with explosives of a moderate power under his directions, and not with high explosives. Black powder may be ordered by him to be used in special cases.

(41) **Surface of Rock Foundation.**—All rock surface intended for masonry foundation must be freed from all loose pieces, and be firm and solid, and prepared as directed by the Engineer.

(42) **Foundation.**—The rock excavation for the foundations is to be extended to such a depth and in such a manner as shall be ordered by the Engineer.

TIMBER WORK.

(43) Timber may be ordered used for platforms, flumes, channels, for permanent sheet-piling and for other permanent uses. It shall be of the size and placed in the manner ordered by the Engineer.

(44) All timber and lumber so used shall be spruce, sound, straight grained and free from all shakes, loose knots and other defects that may impair its strength and durability. Other wood may be accepted by the Engineer if, in his opinion, it is equally good for the particular place in which it is to be used. The price bid for timber shall cover all incidental expenses incurred for labor, or for tools or materials used in placing, securing or fastening it.

(45) No payment shall be made to the Contractor for lumber used for bracing, sheeting, scaffolding and other temporary purposes unless otherwise specified. All timber used for this purpose is to be removed by the Contractor at his own expense, and if any of it is left in the work, no payment shall be made for the same.

(46) **Measurement.**—Only timber work in place is to be measured and estimated. If any round timber is used and accepted, it shall be measured by multiplying the useful length of the stick by the average area of the two finished ends and taking eighty per cent. of the result.

(47) **Tongued and Grooved Timber.**—The timber to be used for sheet-piling in the foundations and other places may be ordered tongued and grooved. Such timber shall be furnished and placed as ordered, and the price herein stipulated (clause O, item (*gg.*)) is to cover the cost of placing, driving, securing and fastening the same. No measurement of the tongues is to be made.

(48) **Crib-work.**—A large amount of crib-work is to be used in connection with the temporary Dams and at other points for the purpose of protecting the work from the water. The cribs will be generally placed in such a position, and constructed in such a manner, as indicated on the plans. They are to be built of tiers of logs or other sound timber acceptable to the Engineer, not more than four feet apart from centre to centre horizontally, and placed vertically above one another. At each point of contact the logs or timber are to be notched into one another and fastened with drift-bolts not less than five-eighths of an inch in diameter, of sufficient length to go through the entire thickness of two contiguous sticks, and into the third to a depth of not less than four inches. No log to be less than twelve inches in diameter at the

large end or less than nine inches at the smaller end. If other timber is used it must not be less than ten inches square in section. Whenever two cribs are placed side by side, as shown on the plans, every other cross log or timber is to extend continuously over the whole of the two cribs, for the purpose of strongly fastening them together. Particular attention must be given to the strength and frequency of this fastening of the two cribs at and near the points of meeting of the crib-work and of the temporary channel, especially on the down-stream side. At and near the ends of the lines of cribs, especially at their connections with the temporary channels, the logs or timbers must be closer to one another, and they must be so shaped and fastened as to make an exact connection.

The cribs are to be filled with stones of as large size as can be accommodated in the timber work, with as much smaller rock as will make a compact filling.

The price herein stipulated (clause O, item (*ggg.*)) per cubic yard of crib-work is to cover the cost of the crib-work complete, in place, including timber, drift-bolts, fastenings, stone filling, and all the materials and labor necessary to make the cribs and to place them in full working order. The work is to measured to the outside lines of each crib, not including the space occupied by the filling of earth between the two cribs.

The space between the two cribs is to be filled with compact material, thoroughly rammed and watered whenever ordered.

Sheeting is to be placed on the various faces of the cribs, of such frequency and depth as, in the opinion of the Engineer, will be rendered necessary by the quality of the materials encountered. If the sheeting cannot be driven to the proper depth, it must be placed in trenches dug for the purpose. On the inside faces of the down-stream crib, especially, a line of deep sheeting shall be placed, to such a depth as the Engineer shall order.

All the excavation necessary for the placing of the cribs and sheeting, all the filling between them, and all the sheeting to be placed in connection therewith, are to be paid for as stipulated in clause O, items (*c.*), (*c c.*), (*d.*) and (*g.*) respectively.

A large amount of stone is to be dumped in front of and over the cribs, after they are completed, especially at their connection with the temporary channel; this work is to be paid for at the price stipulated in clause O, item (*d.*), as specified in clause (35).

MASONRY.

(49) **Hydraulic Masonry.**—All masonry, except where otherwise specified, shall be laid in hydraulic cement mortar, and shall be built of the forms and dimensions shown on the plans, or as directed by the Engineer from time to time, and the system of bonding ordered by the Engineer shall be strictly followed.

(50) All joints must be entirely filled with mortar, and the work in all cases shall be well and thoroughly bonded.

(51) Care must be taken that no water shall interfere with the proper laying of masonry in any of its parts.

(52) All means used to prevent water from interfering with the work, even to the extent of furnishing and placing pipes for conducting the water away from points where it might cause injury to the work, must be provided by the Contractor at his own expense.

(53) Under no circumstances will masonry be allowed to be laid in water.

(54) **Ironwork.**—All ironwork, except the gates, furnished by the City, is to be built in the masonry without other compensation than the price herein stipulated to be paid per cubic yard of masonry.

(55) No masonry is to be built between the 15th of November and the 15th of April, or in freezing weather, except by permission of the Engineer.

(56) All fresh masonry, if allowed to be built in freezing weather, must be covered and protected, and appliances must be procured for heating the water and sand used and for steaming the building materials, all in a manner satisfactory to the Engineer, and during hot weather, all newly built masonry shall be kept wet by sprinkling water on it until it shall have become hard enough to prevent its drying and cracking.

(57) **Cement.**—American cement and Portland cement are to be used. The American cement must be in good condition and must be equal in quality to the best Rosendale cement. It must be made by manufacturers of established reputation, must be fresh and very fine ground, and all cement must be delivered in well-made casks (or equally safe and tight receptacles approved by the Engineer). The Portland cement must be of a brand equal in quality to the best imported Portland cement. To insure its good quality, all the cement furnished by the Contractor will be subject to inspection and rigorous tests; and if found of improper quality, will be branded and must be immediately removed from the work; the character of the tests to be determined by the Engineer. The Contractor shall, at all times, keep in store at some convenient point in the vicinity of the work, a sufficient quantity of cement to allow ample time for the tests to be made without delay to the work of construction. The Engineer shall be notified at once of each delivery of cement. It shall be stored in a tight building, and each cask must be raised several inches above the ground, by blocking or otherwise.

(58) Cement is generally to be used in the form of mortar with an admixture of sand, and when so used, its cost is included in the price herein stipulated for the various kinds of masonry; for the foundation work, however, Portland cement may be ordered by the Engineer to be used without any admixture of sand in exceptionally wet and difficult places, for grouting seams or for such purposes as he may direct; such cement shall be furnished by the Contractor, and if it is used in connection with masonry, it will be paid for in addition to the price herein stipulated to be paid for said masonry. Such cement is to be paid for at so much per barrel of 400 lbs., furnished and delivered by the Contractor at the place where it must be used. See clause O, item (*h.*).

(59) **Mortar.**—All mortar shall be prepared from cement of the quality before described, and clean, sharp sand, free from loam. These ingredients shall be thoroughly mixed dry, as follows: The proportion of cement ordered, by measure, with the ordered proportion of sand, also by measure; and a moderate dose of water is to be afterwards

added to produce a paste of proper consistency; the whole to be thoroughly worked with hoes or other tools. In measuring cement, it shall be packed as received in casks from the manufacturer. The mortar shall be freshly mixed for the work in hand, in proper boxes made for the purpose; no mortar to be used that has become hard or set.

Portland cement is to be used clear for wet work and for pointing, as herein elsewhere specified.

It is also to be used in the proportion of one of cement to two of sand for laying the granite dimension stone, the facing stone, the block stone, masonry and the brick work.

For the rubble stone masonry and concrete masonry, American cement mortar is mostly to be used, and when Portland cement mortar is to be used instead, an additional price is to be paid, as follows:

When Portland cement mortar is to be used for rubble stone masonry and for concrete masonry in the proportion of one part of cement to two parts of sand, an additional price per cubic yard is to be paid to the Contractor, equal to thirty-two per cent. of the price herein stipulated per cubic yard for the said rubble masonry or concrete masonry, respectively (clause O, items (*k.*) and (*i.*))

When Portland cement mortar is to be used for rubble stone masonry in the proportion of one part of cement to three parts of sand, an additional price per cubic yard is to be paid to the Contractor equal to twenty-two per cent. of the price herein stipulated per cubic yard for the said rubble stone masonry (clause O, item (*k.*)).

(60) **Concrete.**—The concrete shall be formed of sound broken stone or gravel not exceeding two inches at their greatest diameter. All stone in any way larger is to be thrown out. The materials to be cleaned from dirt and dust before being used; to be mixed in proper boxes, with mortar of the quality before described, in the proportion of four parts of broken stone to one part of cement; to be laid immediately after mixing, and to be thoroughly compacted throughout the mass by ramming till the water flushes to the surface, the amount of water used for making the concrete to be approved or directed by the Engineer. The concrete shall be allowed to set for twelve hours, or more, if so directed, before any work shall be laid upon it; and no walking over or working upon it shall be allowed while it is setting.

(61) **Bricks.**—The bricks shall be of the best quality of hand-made, hard-burned bricks; burnt hard entirely through, regular and uniform in shape and size, and of compact texture. To insure their good quality, the bricks furnished by the Contractor will be subject to inspection and rigorous tests, and if found of improper quality, will be condemned; the character of the tests to be determined by the Engineer. They are to be culled before laying, at the expense of the Contractor; and all bricks of an improper quality shall be laid aside and removed; the Engineer to be furnished with men for this purpose by and at the expense of the Contractor.

(62) **Brick Masonry.**—All brick masonry shall be laid with bricks and mortar of the quality before described. No "bats" shall be used except in the backing, where a moderate proportion (to be determined by the Engineer) may be used, but nothing smaller than "half-bricks." The bricks to be thoroughly wet just before laying. Every

brick to be completely imbedded in mortar under its bottom and on its sides at one operation. Care shall be taken to have every joint full of mortar.

STONE MASONRY.

(63) All stone masonry is to be built of sound, clean quarry stone of quality and size satisfactory to the Engineer; all joints to be full of mortar, unless otherwise specified.

(64) **Dry Rubble Stone Masonry and Paving.**—Dry rubble masonry and paving are to be laid without mortar, and are to be used for walls, for the slopes of the Dam embankments, and at any other place that may be designated.

(65) This class of masonry is to be of stone of suitable size and quality, laid closely by hand with as few spawls as practicable, in such manner as to present a smooth and true surface. The work is to be measured in accordance with the lines shown on the drawings or ordered during the progress of the work. The stones used must be roughly rectangular; all irregular projection and feather edges must be hammered off. No stone will be accepted which has less than the depth represented on the plans or ordered. Each stone used for paving must be set solid on the foundation of broken stone or earth and no interstices must be left.

(66) In the dry rubble masonry walls, large stones must be used, especially for the faces, and the walls must be bonded with frequent headers, of such frequency and sizes as shall be approved by the Engineer.

(67) **Rip-rap.**—Rip-rap may be used in connection with the protective work, and wherever the Engineer may order it. It shall be made of stone of such size and quality and in such manner as he shall direct, and must be laid by hand.

(68) **Broken Stones.**—After the slopes which are to receive the paving have been dressed, a layer of broken stone is to be spread as a foundation for the paving, wherever ordered. The broken stones must be sound and hard, not exceeding two inches at their greatest diameter. Broken stone, not exceeding one inch in diameter, may be used for forming roadways; it is to be spread to such thickness as ordered and heavily rolled or rammed. Broken stones may be used also wherever the Engineer may direct, rolled if so directed, and paid for under this head, except the broken stone used for making concrete, the cost of which is included in the price hereinbefore stipulated for concrete laid.

(69) **Rubble Stone Masonry.**—Rubble stone masonry is to be used for the central part of the Dam, for the overflow, for the centre walls of the earth embankments, for most of the structures and appurtenances of the Dam, and wherever ordered by the Engineer.

Rubble stone masonry shall be made of sound, clean stone of suitable size, quality and shape for the work in hand, and presenting good beds for materials of that class. Especial care must be taken to have the beds and joints full of mortar, and no grouting or filling of joints after the stones are in place will be allowed. The work must be thoroughly bonded. The faces of the rubble stone masonry, especially the up-stream face of the walls, shall be closely inspected after they are built, and if any mortar

joints are not full and flush, they shall be taken out to a depth of no less than three inches or more, if so ordered, and repointed properly.

(70) **Central Part of the Dam and Overflow.**—A large quantity of rubble stone masonry in mortar is to be used in the construction of the central part of the Dam and of the centre wall and overflow.

The stones used therein must be sound and durable; they must have roughly rectangular forms, and all irregular projections and feather edges must be hammered off. Their beds, especially, must be good for materials of that class, and present such even surfaces that, when lowering a stone on the level surface prepared to receive it, there can be no doubt that the mortar will fill all spaces. After the bed joints are thus secured, a moderate quantity of spawls can be used in the preparation of suitable surfaces for receiving other stones. All other joints must be equally well filled with mortar.

The quality of the beds is to regulate, to a large extent, the size of the stones used, as the difficulty of forming a good bed joint increases with the size of the stones.

Various sizes must be used, and regular coursing must be avoided, in order to obtain vertical as well as horizontal bonding.

(71) **Sizes.**—The sizes of the stones used will vary also with the character of the quarries, but, especially in the places where the thickness of masonry is great, a considerable proportion of large stones is to be used. If the size and character of the stones in the opinion of the Engineer shall admit of it, the joints (except the beds), instead of being filled with mortar, may, at his request or on his approval, be filled with concrete made as hereinbefore specified, with the exception that the component materials shall be mixed in the proportion of one part of cement to three parts of small stone or gravel of such size as the Engineer shall direct, and thoroughly rammed, care being taken to use a moderate amount of water only which must be brought to the surface by ramming, such filling of joints with concrete to leave no vacancies and to be thoroughly made. If concrete is so used, the spaces left between the stones should not be less than six inches, in order that proper ramming can be obtained.

No extra compensation shall be paid to the Contractor for the use of such concrete, the cost of which is to be included in the price herein stipulated for the masonry in connection with which it is used.

(72) **Face Work for Rubble Stone Masonry.**—The exposed faces of the main wing wall, of road culverts, of some of the walls and of any other rubble work that the Engineer may designate, are to be made of broken ashlar with joints not exceeding one-half inch in thickness; the stones not to be less than 24 inches deep from the face, and to present frequent headers. This face work to be equal in quality and appearance to the face of the breast wall in front of the new Gate House at Croton Dam, and to be well pointed with Portland cement. This face work is to be paid for by the square foot of the superficial area for which it is ordered, in addition to the price paid per cubic yard of rubble stone masonry.

(73) **Block Stone Masonry.**—Block stone masonry is to be composed mainly of large blocks and is to be used for the steps of the overfall or for other steps, or whenever and wherever ordered by the Engineer. It is to be laid in Portland cement

mortar, well pointed, or may be ordered laid dry at the price stipulated in clause O, item (*o.*).

This stone, which is to receive the shock of water and ice, is to be especially sound, hard and compact, and of a durable character; it is to be prepared to the dimensions given so that no joint will in any place be more than one inch wide. The outside arrises must be pitched to a true line.

(74) **Facing Stone Masonry.**—The outer faces of the Masonry Dam and of its gate chambers, of the overflow, (except steps,) and of any other piece of masonry that may be designated, are to be made of range stones, as shown on the plans, the stone to be of unobjectionable quality, sound and durable, free from all seams, discoloration and other defects, and of such kind as shall be approved by the Engineer.

(75) All beds, builds and joints are to be cut true to a depth of not more than 4 inches, and not less than 3 inches from the faces to surfaces, allowing of one-half inch joints at most; the joints for the remaining part of the stones not to exceed two inches in thickness at any point.

(76) All cut arrises to be true, well defined and sharp.

(77) Where this class of masonry joins with granite dimension stone masonry the courses must correspond, and the joining with arches and other dimension stone masonry must be accurate and workmanlike.

Each course to be composed of two stretchers and one header alternately, the stretchers not to be less than 3 feet long nor more than 7 feet long, and the headers of each successive course to alternate approximately in vertical position.

(78) The rise of the courses may vary from bottom to top from 30 inches to 15 inches in approximate vertical progression, and the width of bed of the stretchers is not to be at any point less than 28 inches. The headers are not to be less than 4 feet in length.

This class of masonry, for the faces of the Dam and gate chamber, including the headers, is to be estimated at 30 inches thick throughout. At other places that may be designated by the Engineer, the size of the stones is to be established by him, and the facing stone masonry is to be estimated according to the lines ordered or shown on the plans. In no case are the tails of the headers to be estimated.

The work to be equal in quality and appearance to the facing stone masonry work built by the Aqueduct Commissioners for the Masonry Dam across the East Branch of the Croton River near Brewster.

(79) All copings that may be ordered and the heads of the arches of the highway culverts, will be classed as facing stone masonry.

(80) The price herein stipulated for facing stone masonry is to cover the cost of pointing, of cutting chisel drafts at all corners of the Gate House Dam and other corners, and of preparing the rock faces; but if any six-cut or rough-pointed work is ordered in connection with this class of masonry it shall be paid for at the prices herein stipulated for such work. Clause O, items (*s.*) and (*t.*).

(81) The face bond must not show less than 12 inches lap, unless otherwise permitted.

(82) The pointing of the faces to be thoroughly made with pure Portland cement

after the whole structure is completed; unless otherwise permitted, every joint to be raked out therefor to a depth of at least two inches, and if the Engineer is satisfied that the pointing at any place is not properly made it must be taken out and made over again.

(83) **Granite Dimension Stone Masonry.**—Granite dimension stone masonry must be made of first-class granite of uniform color, free from all seams, discoloration and other defects, and satisfactory to the Chief Engineer.

(84) It is to be used for the gate openings in the gate chamber, for the coping of the Dam, for the Gate House superstructures and for the crest and first step of the overflow, and at any other place that may be designated by the Engineer.

(85) The stones shall be cut to exact dimensions, and all angles and arrises shall be true, well defined and sharp.

(86) All beds, builds and joints are to be dressed, for the full depth of the stone, to surfaces, allowing of one-quarter ($\frac{1}{4}$) inch joint at most. No plug-hole of more than 6 inches across or nearer than 3 inches from an arris is to be allowed, and in no case must the aggregate area of the plug-hole in any one joint exceed one-quarter of its whole area.

(87) The stone shall be laid with one-quarter ($\frac{1}{4}$) inch joints, and all face joints shall be pointed with mortar made of clear Portland cement, applied before its first setting. All joints to be raked out to a depth of two inches before pointing.

(88) **Pointing.**—The pointing of all masonry, including the faces of the main body of the Dam and of the centre walls which are below the ground, is to be done thoroughly with Portland cement mortar, mixed clear where used for all exposed faces of brick and cut stone masonry of all kinds (including the rubble facing); and mixed for other work in such proportion as the Engineer shall determine. The cost of all pointing is to be included in the price stipulated for the masonry to which it is applied.

(89) **Face Dressing.**—The exposed faces of the cut stone are to be finished in various ways, in accordance with the various positions in which they are placed. They shall be either left with a rock or quarry face, rough-pointed, or fine hammered (six-cut work).

(90) The various classes of face dressing must be equal in quality and appearance to those on the sample in the office of the Chief Engineer.

(91) **Rock Face Dressing.**—In rock face work the arrises of the stones inclosing the rock face must be pitched to true lines; the face projections to be bold, and from 3 to 5 inches beyond the arrises. The angles of all walls on structures having rock faces are to be defined by a chisel draft not less than $1\frac{1}{2}$ inches wide on each face.

(92) **Rough-pointed Dressing.**—In rough-pointed work, the stones shall at all points be full to the true plane of the face, and at no point shall project beyond more than $\frac{1}{4}$ inch, the arrises to be sharp and well defined. Each stone to have its arrises well defined by a chisel draft, which is included in the price for rough-pointed dressing.

(93) **Fine Hammered (six-cut) Dressing.**—In fine hammered work the face of the stones must be brought to a true plane and fine dressed, with a hammer having six blades to the inch.

(94) In measuring cut stone masonry, when the stones are not rectangular, the

dimensions taken for each stone will be those of a rectangular, cubical form which will just inclose the neat lines of the same. The price herein stipulated for granite dimension stone masonry is to cover the cost of preparing the rock faces, of making the chisel drafts, and of preparing all holes and recesses and grooves.

(95) No payment will be made for cutting grooves and recesses other than the price paid for the dressing of their surfaces, which are to be fine hammered.

(96) For rough-pointed and fine hammered (six-cut) dressing, a price per square foot of dressing will be paid in addition to the price per cubic yard of masonry, viz.:

(97) For rough-pointed dressing, the price stipulated in clause O, item (*t*.), and for fine hammered (six-cut) dressing, the price stipulated in clause O, item (*s*.).

(98) The exposed parts of the cut stone are generally to be prepared with rock face.

(99) The inside surfaces and copings are generally to be rough-pointed.

(100) All the gateways, grooves, sills, floors, and all other surfaces designated by the Engineer are to be fine hammered.

IRONWORK.

(101) All ironwork, such as spikes, bolts for fastening timber work, pipes, and all other ironwork used for temporary purposes, is to be furnished by the Contractor at his own expense.

All other ironwork which is to be used for permanent purposes, such as dowels for stonework, cast-iron pipes, beams, wall castings, ladders, and all other necessary ironwork which is to be built into the masonry, is to be furnished by the City and delivered in the vicinity of the work, but the Contractor is to haul them into place and to build them in the masonry at his own expense, including lead and other necessary materials, under the direction of the Engineer. Many parts of that ironwork are large and heavy and the placing of them will reouire considerable time and care.

When the Old Croton Aqueduct is discontinued for the purpose of connecting with the work of the Dam, a pipe is to be laid by the City for the purpose of uniting the two disconnected ends, and the Contractor is to do, at such time as the Engineer shall indicate, all necessary excavation and refilling necessary therefor, and to give the City all facilities for the work.

SUPERSTRUCTURE OF GATE HOUSES.

(102) The masonry part of the Superstructure of the Gate House is to be erected under this contract. The base, cornices, window and door trimmings are to be built of granite dimension stone masonry with such surface cutting as shall be ordered; the walls shall be built of the same stone, variously faced. The general appearance and style of the buildings will be similar to those of the new Gate House near the present Croton Dam; but through stones with two faces shall be used for the walls instead of brick and stone.

FENCING.

(103) Fencing must be erected along the new highway wherever ordered; it is to be erected on the same plan as adopted by the City for the new highways built around the Brewsters Reservoir with the addition of one horizontal rail, and painted in the same manner.

GENERAL CLAUSES.

(104) **Old Croton Aqueduct.**—The Contractor is to remove a portion of the Old Croton Aqueduct; as much of it and at such time as the Engineer shall designate, and the work of reconstruction of the same, in connection with the Dam, must proceed continuously and without delay. The Contractor must so arrange his work as not to interfere with the flow of water.

(105) **Fences.**—If found necessary, the Contractor shall erect and maintain, at his own expense, fences, for the protection of the adjoining property.

(106) **Cleaning and Finishing.**—At his own expense, and under the direction of the Engineer, the Contractor is to clear the work and the grounds occupied by him from all refuse and rubbish, and to leave them in neat condition.

(107) **Facilities.**—The Contractor is to give all facilities to the City for performing work which may be adjoining his own, or connected therewith.

(108) **Contractor not Present.**—Whenever the Contractor is not present on any part of the work where it may be necessary to give directions, orde s will be given by the Engineer to, and shall be received and obeyed by, the superintendent or overseer of the Contractor who may have charge of the particular work in relation to which the orders are given.

(109) **Force to be Employed.**—The work shall be commenced and carried on in such order and at such times, points and seasons, and with such force as may, from time to time, be directed by the Engineer.

(110) **Lines and Grades.**—All lines and grades are to be given by the Engineer, who may change them from time to time as he may be authorized and directed by the said Aqueduct Commissioners.

(111) **Marks and Stakes.**—The stakes and marks given by the Engineer must be carefully preserved by the Contractor, who must give to the Engineer all necessary assistance and facilities for establishing the benches and plugs, and for making measurements.

(112) **Engineer to Explain Specifications.**—The plans and specifications are intended to be explanatory of each other, but should any discrepancy appear, or any misunderstanding arise as to the import of anything contained in either, the explanation of the Engineer shall be final and binding on the Contractor; and all directions and explanations required, alluded to, or necessary to complete any of the provisions of these specifications, and give them due effect, will be given by the Engineer.

(113) Any unfaithful or imperfect work that may be discovered before the final acceptance of the work shall be corrected immediately on the requirement of the

Engineer, notwithstanding that it may have been overlooked by the proper Inspector and estimated.

The inspection of the work shall not relieve the Contractor of any of his obligations to perform sound and reliable work, as herein described. And all work of whatever kind which, during its progress and before it is finally accepted, may become damaged for any cause, shall be broken up or removed, so much of it as may be objectionable, and replaced by good and sound work, satisfactory to the Engineer.

NOTE A.

(*See page* 33.)

Maximum value for

$$\frac{u}{x} = 1 - \frac{2(d^3 + b^3)}{3(d^3 + db^2)}, \quad \ldots \ldots \ldots \quad (1)$$

in which $d \geqq b$.

The maximum value will occur when $\dfrac{d^3 + b^3}{d^3 + db^2}$ is a minimum.

Finding the First Differential Coefficient of this fraction, and placing it equal to 0, we obtain

$$\frac{2d^3 - 3d^2b - b^3}{(d^3 + db^2)^2} = 0. \quad \ldots \ldots \ldots \quad (2)$$

The Second Differential Coefficient is obtained next, and found to be positive; hence any value for d which will satisfy Equation (2) will give a minimum value for the fraction, and consequently a maximum value for $\dfrac{u}{x}$.

Solving Equation (2) by Cardan's formula, we find

$$d = 1.677648b,$$

which is the depth corresponding to the maximum value of $\dfrac{u}{x}$.

Substituting this value of d in Equation (1), we find the maximum value of

$$\frac{u}{x} = 0.40392.$$

NOTE B.

(*See page* 34.)

Minimum value for

$$\frac{n}{x} = \frac{d^3 + 2b^3}{3(d^3 + db^2)}, \quad \ldots \ldots \ldots \quad (1)$$

in which $d \geqq b$.

Finding the First Differential Coefficient of this fraction, omitting the constant factor $\frac{1}{3}$, and placing it equal to 0, we obtain

$$\frac{d^3 - 3d^2b - b^3}{(d^2 + db^2)^2} = 0. \quad \ldots \ldots \ldots \quad (2)$$

The Second Differential Coefficient is derived next, and found to be positive; hence any value for d which will satisfy Equation (2) will give a minimum value for the fraction $\dfrac{n}{x}$.

Solving Equation (2) by Cardan's formula, we find

$$d = 3.1038b,$$

which is the depth corresponding to a minimum value of $\frac{n}{x}$.

Substituting this value of d in Equation (1), we find the minimum value of

$$\frac{n}{x} = 0.32218.$$

NOTE C.

(*See page* 37.)

The stability of Practical Profile No. 3 *has been found graphically* (see Plate XX.) *by Mr. G. Bonanno, C.E. and Arch.*, who has also demonstrated that this profile presents at all depths a factor of safety of at least 2 against overturning. The following method was employed:

Reservoir Empty.—Profile $MNQP$ (Fig. 1) is divided into 10 courses, 1, 2, 3, ... 10, by the horizontal joints $aa, bb, cc, \ldots\ldots kk$. The centres of gravity, $g_1, g_2, g_3, \ldots\ldots g_{10}$, of courses 1 to 10 are found in the usual manner, the vertical section of each course being assumed to form a trapezoid or rectangle. The centres of gravity, $g_1, g_2, g_3, \ldots\ldots g_{10}$, and the centre points of the joints $aa, bb, cc, \ldots\ldots kk$, are connected by a line which will be the *medial line* of the profile $MNQP$.

Find next for each joint, $aa, bb, cc, \ldots\ldots kk$, the resultant of the forces acting in the wall above it. The points where these resultants intersect their respective joints will lie in the *line of pressure*. When the reservoir is empty, the only forces acting on the wall are the weights of the different courses 1 to 10, each being applied at the centre of gravity of the respective course. Through $g_1, g_2, g_3, \ldots\ldots g_{10}$ draw the vertical lines 1–1, 2–2, 3–3, ... 10–10 (Fig. 3). On the vertical line OZ (Fig. 2) of the force polygon draw to any convenient scale $Ow_1, Ow_2, Ow_3, \ldots\ldots Ow_{10}$, equal to the weights above the joints $aa, bb, cc, \ldots\ldots kk$, respectively. Assuming any point P as a pole in the force polygon (Fig. 2), we can obtain the corresponding funicular polygon (Fig. 3) which gives for each joint the position of the resultant of the weights resting on it. Thus for the joint hh the value of the resultant is given by OW_8 (Fig. 2), and its position is determined by the point S_8, where the lines drawn parallel to PO and PW_8 in the funicular polygon (Fig. 3) intersect. The point S_8' (Fig. 1) at which a vertical line (W_8) passing through S_8 intersects the joint hh is one point in the *line of pressure, reservoir empty*. In a similar manner the points where this line intersects the other joints were found.

Reservoir Full.—The position of the *line of pressure, reservoir full*, can be determined as follows:

Lay off in the force polygon on the horizontal line OH the distances $Ot_1, Ot_2, Ot_3, \ldots\ldots Ot_{10}$, which represent the horizontal thrusts of the water for the joints $aa, bb, cc, \ldots\ldots kk$, respectively. (The vertical component of the water-pressure is neglected as in the Analytical Method given in Chapter III.) The lines $w_1t_1, w_2t_2, w_3t_3, \ldots\ldots w_{10}t_{10}$ will represent the resultants acting on the joints $aa, bb, cc, \ldots\ldots kk$ when the reservoir is full. To find the points where these resultants are applied to their respective joints, draw horizontal lines through the points $r_1, r_2, r_3, \ldots\ldots r_{10}$, through which pass the resultant water-pressures

for the vertical depths: Na, Nb, Nc, ... NQ. The points r_1, r_2, r_3, ... r_{10} are at $\frac{2}{3}$ of the vertical depths of their respective joints below the top of the dam.

The horizontal lines through r_1, r_2, r_3, ... r_{10} are produced until they intersect the vertical lines W_1, W_2, W_3, ... W_{10}, which represent the resultant pressures of the weights of the courses on the joints aa, bb, cc, ... kk. From the intersection points found thus draw the lines R_1, R_2, R_3, ... R_{10} parallel to t_1w_1, t_2w_2, t_3w_3, ... $t_{10}w_{10}$, and produce them until they intersect the corresponding joints aa, bb, cc, ... kk. For instance, the thrust of the water above the joint hh passes through r_8, through which a horizontal line is produced until it meets the vertical line W_8 in V. From this point draw a line parallel to t_8w_8 (Fig. 2). The point V' at which it intersects the joint hh is a point in the *line of pressure, reservoir full.* In a similar manner the points where this line crosses the other joints may be found.

Factor of Safety against Overturning.—To test whether the profile $MNQP$ has at all depths a factor of safety of 2 against overturning, imagine the reservoir to be filled with a liquid having double the density of water and exerting, therefore, twice the horizontal pressure of water. The line of pressure for this assumed liquid is found in precisely the same manner as for water. In the force polygon (Fig. 2) lay out the new horizontal pressures in the line OH. Draw dotted lines (see Fig. 2) from the points w_1, w_2, w_3, ... w_{10} to the corresponding points in the line OH. The dotted lines will represent the new resultants. From the points where the horizontal lines through r_1, r_2, r_3 . . . . . . r_{10} meet the vertical lines W_1, W_2, W_3 . . . . . . W_{10} (Fig. 1) draw lines respectively parallel to the dotted lines (Fig. 2). The points where these lines intersect the corresponding joints aa, bb, cc, ... kk are points on the line of pressure for a liquid having a specific gravity of 2. The fact that this line lies entirely within the profile $MNQP$ proves that this profile will ensure at all depths a factor of safety of 2 against overturning.

The resistance which this profile offers against sliding or shearing is found by measuring the angles which the resultants (reservoir full) make with vertical lines. Thus for the joint hh the angle to be measured is that between R_8 and W_8. So long as the tangent of these angles is less than 0.75 (see page 15), the dam will have ample strength at the corresponding joints against sliding.

The maxima pressures at any joint can be found by measuring the distances between the line of pressures, reservoir full and empty, ~~with~~ and the front and back face respectively, which distances must be substituted in formula (A) or (B) (page 10).

TABLES.

The following Tables, which have been referred to in the previous pages, serve to illustrate our text by giving: (1) The dimensions and strength of the profile-types designed by eminent engineers; (2) Proofs of the principles discussed in this book; (3) Practical examples of the method we have proposed for designing profiles.—The peculiar top width (18.74 feet) adopted for some of our profiles is that assumed by Prof. Rankine for his Logarithmic Type (see Table III.), and was used by us for comparison. Table XXIV. facilitates the reduction of metres into feet, and gives the English equivalent of the metric square and cubic measures.

TABLE I.*

(See page 2.)

M. DE SAZILLY'S PROFILE-TYPE.

Depth of Water, in Metres.	Horizontal Thrusts, in Tons of 2205 lbs.	Total Vertical Pressures, in Tons of 2205 lbs.	Area, in Square Metres.	Distance to Line of Pressure.		Maximum Pressure to the Square Centimetre.		Coefficient of Friction necessary for Equilibrium.
				From Front Face, Reservoir Full.	From Back Face, Reservoir Empty.	Reservoir Full.	Reservoir Empty.	
9	40.50	90.00	45.00	1.15	2.50	5.22	1.80	0.450
11	60.50	113.20	56.60	1.25	2.58	6.00	2.60	0.534
13	84.50	140.88	70.44	1.56	2.75	6.00	3.28	0.599
15	112.50	173.56	86.78	1.92	3.00	6.01	3.81	0.648
17	144.50	211.72	105.86	2.35	3.32	6.02	4.24	0.682
19	180.50	255.76	127.88	2.84	3.70	6.00	4.61	0.705
21	220.50	306.04	153.02	3.40	4.12	6.00	4.95	0.720
23	264.50	362.88	180.44	4.03	4.59	6.00	5.27	0.729
25	312.50	426.64	213.32	4.74	5.09	6.00	5.58	0.732
27	364.50	497.68	248.84	5.53	5.63	6.00	5.89	0.732
29	420.50	579.98	288.44	6.47	6.42	6.00	5.99	0.725
31	480.50	674.23	332.52	7.57	7.39	6.01	6.00	0.713
33	544.50	780.25	381.42	8.81	8.47	6.00	6.00	0.698
35	612.50	898.69	435.36	10.13	9.65	6.00	5.98	0.682
37	684.50	1030.33	494.62	11.56	10.93	6.00	5.96	0.664
39	760.50	1174.10	559.38	13.06	12.22	5.98	5.99	0.648
41	840.50	1333.24	629.98	14.67	13.65	5.98	6.00	0.630
43	924.50	1507.79	706.70	16.38	15.16	5.97	6.00	0.613
45	1012.50	1698.58	789.84	18.20	16.77	5.98	6.00	0.596
47	1104.50	1906.43	879.70	20.11	18.48	5.98	6.00	0.579
49	1200.50	2132.19	976.60	22.12	20.27	5.98	6.00	0.563
51	1300.50	2377.29	1080.00	24.27	22.18	6.00	6.00	0.547

Specific Gravity of the Masonry = 2. 1 Ton = 1 cubic metre of water = 2204.737 lbs.

* From M. Delocre's memoir in the "Annales des Ponts et Chaussées," 1866.

TABLE II.*

(See page 3.)

M. DELOCRE'S PROFILE-TYPE.

Depth of Water, in Metres.	Horizontal Thrusts, in Tons of 2205 lbs.	Total Vertical Pressures, in Tons of 2205 lbs.	Area, in Square Metres.	Distance to Line of Pressure.		Maximum Pressure to the Square Centimetre.		Coefficient of Friction necessary for Equilibrium.
				From Front Face, Reservoir Full.	From Back Face, Reservoir Empty.	Reservoir Full.	Reservoir Empty.	
2	2.00	20.51	10.253	2.62	2.56	0.39	0.27	0.095
4	8.00	42.02	21.012	2.59	2.65	0 92	0.85	0.190
6	18.00	64.56	32.278	2.50	2.69	1.57	1.34	0.275
8	32.00	88.10	44.052	2.28	2.76	2.52	1.84	0.360
10	50.00	112.66	56.330	1.94	2.82	3.86	2.34	0.440
12	72.00	138.24	69.120	1.54	2.89	5.98	2.83	0.520
14	98.00	167.22	83.608	2.21	3.02	5.05	3.65	0.585
16	128.00	201.99	100.993	2.78	3.25	4.84	4.16	0.630
18	162.00	242.55	121.275	3.30	3.56	4.90	4.54	0.665
20	200.00	288.91	144.454	3.78	3.92	5.10	4.92	0.690
22	242.00	341.06	170.530	4.24	4.32	5.36	5.27	0.705
24	288.00	399.01	199.503	4.69	4.74	5.67	5.61	0.720
26	338.00	462.76	231.380	5.14	5.19	6.00	5.95	0.730
28	392.00	548.85	267.020	6.31	6.16	5.79	5.79	0.714
30	450.00	645.24	307.312	7.43	7.21	5.81	5.68	0.697
32	512.00	761.80	357.247	8.57	8.23	5.85	5.73	0.672
34	578.00	869.12	401.828	9.76	9.26	5.89	5.76	0.665
36	648.00	996.65	456.055	10.83	10.28	5.99	5.87	0.640
38	722.00	1134.59	514.940	12.12	11.30	6.00	6.00	0.636
40	800.00	1307.07	579.283	13.89	12.90	5.96	5.90	0.612
42	882.00	1494.37	649.912	15.69	14.48	5.91	5.85	0.589
44	968.00	1696.50	726.830	17.46	16.04	5.93	5.81	0.570
46	1058.00	1913.41	810.020	19.17	17.60	5.94	5.85	0.552
48	1152.00	2145.19	899.515	21.00	19.14	5.93	5.98	0.537
50	1250.00	2391.78	995.300	22.76	20.67	6.00	6.00	0.522

Specific Gravity of the Masonry = 2. 1 Ton = 1 cubic metre of water = 2204.737 lbs.

* From M. Delocre's memoir in the "Annales des Ponts et Chaussées," 1866.

TABLE III.

(See page 28.)

PROF. RANKINE'S PROFILE-TYPE.

Depth of Water Below Top of Dam in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0	0	0	17.40	1.34	18.74	0	9.37	9.37	0.0	0.00	0.0	0.00	0.00	0.0
10	25	83	19.72	1.52	21.24	200	10.73	10.09	9.1	0.57	10.8	0.68	0.13	26.7
20	100	667	22.35	1.72	24.07	426	11.62	10.89	19.6	1.23	22.8	1.43	0.23	8.5
30	225	2250	25.29	1.94	27.23	679	12.13	11.79	33.1	2.07	34.9	2.18	0.33	4.7
40	400	5333	28.69	2.21	30.90	973	12.57	12.85	49.1	3.07	47.4	2.96	0.41	3.3
50	625	10417	32.53	2.50	35.03	1303	13.01	14.02	65.9	4.12	59.5	3.72	0.48	2.6
60	900	18000	36.83	2.83	39.66	1674	13.56	15.35	82.2	5.14	70.8	4.43	0.54	2.3
70	1225	28583	41.75	3.21	44.96	2098	14.48	16.86	96.6	6.04	81.7	5.11	0.58	2.1
80	1600	42667	47.31	3.64	50.95	2577	15.83	18.57	108.5	6.78	91.7	5.73	0.62	2.0
90	2025	60750	53.61	4.12	57.73	3119	17.74	20.51	117.2	7.33	100.9	6.31	0.65	1.9
100	2500	83333	60.75	4.67	65.42	3734	20.39	22.71	122.1	7.63	109.5	6.84	0.67	1.9
110	3025	110917	68.84	5.29	74.13	4431	23.91	25.19	123.6	7.73	117.3	7.33	0.68	2.0
120	3600	144000	78.00	6.00	84.00	5221	28.40	28.02	122.6	7.66	124.3	7.77	0.69	2.0
130	4225	183083	88.39	6.80	95.19	6116	34.05	31.21	119.1	7.44	130.6	8.16	0.69	2.1
140	4900	228667	100.15	7.70	107.85	7129	40.95	34.83	113.8	7.11	136.5	8.53	0.69	2.3
150	5625	281250	113.49	8.73	122.22	8278	49.31	38.94	107.0	6.69	141.7	8.87	0.68	2.5
160	6400	341333	128.60	9.90	138.50	9581	59.29	43.59	99.1	6.19	146.5	9.16	0.67	2.7
170	7225	409417	145.72	11.21	156.93	11055	71.05	48.85	90.3	5.64	150.9	9.43	0.65	2.9
180	8100	486000	165.14	12.70	177.84	12728	84.83	54.82	81.4	5.09	154.8	9.68	0.64	3.2
190	9025	571583	187.10	14.40	201.50	14621	100.82	61.59	72.6	4.54	158.3	9.89	0.62	3.6
200	10000	666667	212.00	16.30	228.30	16765	119.30	69.24	63.6	3.98	161.4	10.09	0.60	4.0

The Specific Gravity of the Masonry = 2.

TABLE IV.

(See page 28.)

THEORETICAL PROFILE.

BASED ON PROFESSOR RANKINE'S LIMITS AND CONDITIONS.

Depth of Water Below Top of Dam in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area in Square Feet.	Distance from Front Face to Line of Pressure. Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.0	0	0	18.74	0.00	18.74	0	9.37	9.37	0.0	0.00	0.0	0.00	0.00	0.0
26.5	175	1551	18.74	0.00	18.74	497	6.25	9.37	53.0	3.31	26.5	1.66	0.35	3.0
40.0	400	5333	25.08	0.00	25.08	792	8.36	9.99	63.1	3.94	50.8	3.18	0.51	2.2
50.0	625	10417	31.17	0.00	31.17	1074	10.39	11.08	68.9	4.31	64.4	4.03	0.58	2.1
60.0	900	18000	37.92	0.00	37.92	1419	12.64	12.60	74.8	4.68	75.1	4.69	0.63	2.0
70.0	1225	28583	45.52	1.04	46.56	1842	15.52	15.52	79.1	4.94	79.1	4.94	0.66	2.0
80.0	1600	42667	52.83	1.64	54.47	2347	18.16	18.16	86.2	5.39	86.2	5.39	0.68	2.0
90.0	2025	60750	60.16	2.04	62.20	2930	20.73	20.73	94.2	5.89	94.2	5.89	0.69	2.0
100.0	2500	83333	67.36	2.29	69.65	3589	23.22	23.22	103.1	6.44	103.1	6.44	0.69	2.0
110.0	3025	110917	74.52	2.46	76.98	4322	25.66	25.66	112.3	7.02	112.3	7.02	0.70	2.0
120.0	3600	144000	81.65	2.58	84.23	5129	28.08	28.08	121.8	7.61	121.8	7.61	0.70	2.0
130.0	4225	183083	90.36	3.38	93.74	6018	32.07	31.25	125.0	7.81	128.4	8.03	0.70	2.1
140.0	4900	228667	100.24	4.53	104.77	7011	37.23	34.92	125.0	7.81	133.8	8.36	0.70	2.1
150.0	5625	281250	110.55	5.64	116.19	8116	42.81	38.73	125.0	7.81	139.7	8.73	0.69	2.2
160.0	6400	341333	121.28	6.72	128.00	9337	48.78	42.66	125.0	7.81	145.9	9.12	0.69	2.3
170.0	7225	409417	132.40	7.78	140.18	10678	55.12	46.72	125.0	7.81	152.3	9.52	0.68	2.4
180.0	8100	486000	143.91	8.82	152.73	12142	61.80	50.90	125.0	7.81	159.0	9.94	0.67	2.5
190.0	9025	571583	156.16	11.91	168.07	13746	69.25	57.23	125.0	7.81	160.0	10.00	0.66	2.6
200.0	10000	666667	168.93	15.86	184.79	15510	77.32	64.49	125.0	7.81	160.0	10.00	0.65	2.8

The Specific Gravity of the Masonry = 2.

TABLE V.

(See page 6.)

M. KRANTZ'S PROFILE-TYPE.

Depth of Water, in Metres	Vertical Water-pressure, in Cubic Metres of Masonry.	Horizontal Thrust of Water, in Cubic Metres of Masonry.	Moment of Water, in Cubic Metres of Masonry.	Joint Referred to a Vertical Axis. Left of Axis, in Metres.	Right of Axis, in Metres.	Total, in Metres.	Total Area, in Square Metres.	Distance from Front Face to Line of Pressure, Reservoir Full, in Metres.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Metres.	Maxima Pressures. Reservoir Full. In Metres of Masonry.	In Kilos perSq. Cent.	Reservoir Empty. In Metres of Masonry.	In Kilos perSq. Cent.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
0	0.00	0.00	0.00	5.00	0.00	5.00	17.50	2.50	2.50	7.00	1.61	7.00	1.61	0.00	0.0
5	0.20	5.43	9.06	5.29	0.14	5.43	43.21	2.60	2.64	8.95	2.06	8.59	1.98	0.12	13.4
10	1.59	21.74	72.46	6.16	0.55	6.71	73.19	2 67	3.13	18.05	4.15	13.09	3.01	0.29	3.8
15	5.41	48.91	244.57	7.65	1.24	8.89	111.74	3.00	3.98	26.09	6.00	16.59	3.82	0.42	2.4
20	12.95	86.96	579.71	9.84	2.23	12.07	163.83	3.90	5.21	30.22	6.95	19.00	4.37	0.49	2.2
25	25.46	135.87	1132.24	12.85	3.50	16.35	234.33	5.65	6.87	30.51	7.02	21.21	4.88	0.52	2.3
30	44.51	195.65	1956.52	16.91	5.09	22.00	329.60	8.60	9.00	28.23	6.49	23.07	5.31	0.52	2.6
35	71.57	266.30	3106.89	22.50 (23.50)	7.00 (8.00)	29.50 (31.50)	457.31	13.13	11.71	24.03	5.53	25.11	5.78	0.50	3.2
40	141.10	347.83	4637.69	28.50	11.33	39.83	635.61	19.09	17.01	21.84	5.02	22.98	5.29	0.45	4.2
45	202.73	440.22	6603.26	33.50	14.67	48.17	855.61	23.62	21.21	23.29	5.35	24.15	5.55	0.41	4.8
50	271.58	543.48	9059.97	38.50	18.00	56.50	1117.31	28.16	25.31	25.07	5.77	26.10	6.00	0.39	5.3

The Specific Gravity of the Masonry = 2.3.

TABLE VI.

(See page 6.)

PROF. A. R. HARLACHER'S PROFILE-TYPE.

Position of Horizontal Joint Below Top of Dam, in Metres.	Depth of Water, in Metres.	Horizontal Component of Water-Pressure, in Tons of 1000 Kilos.	Vertical Component of Water-Pressure, in Tons of 1000 Kilos.	Joint Referred to Vertical Axis. Left of Axis, in Metres.	Right of Axis, in Metres.	Total, in Metres.	Total Area of Profile, in Square Metres.	Total Weight of Masonry, in Tons of 1000 Kilos.	Distance from Front Face to Line of Pressure, Reservoir Full, in Metres.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Metres.	Maxima Pressures. Reservoir Full. In Kilos. per Sq. Cent.	Reservoir Empty. In Kilos. per Sq. Cent.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
0.00	0.00	0.0	0.0	4.00	0.00	4.00	0.00	0.0	2.00	2.00	0.00	0.00	0.00	0.0
2.50	0.00	0.0	0.0	4.00	0.00	4.00	10.00	22.0	2.00	2.00	0.55	0.55	0.00	0.0
5.00	2.50	3.1	0.0	4.00	0.00	4.00	20.00	44.0	1.94	2.00	1.12	1.10	0.07	33.3
7.50	5.00	12.5	0.0	4.20	0.00	4.20	30.20	66.4	1.87	2.10	2.10	1.58	0.19	6.9
10.00	7.50	28.1	0.0	4.75	0.00	4.75	41.35	90.9	1.87	2.12	3.11	2.55	0.31	3.4
12.50	10.00	50.0	0.0	5.65	0.00	5.65	54.35	119.5	2.02	2.22	3.90	3.43	0.42	2.4
15.00	12.50	78.1	0.0	7.05	0.00	7.05	70.15	154.3	2.47	2.47	4.13	4.16	0.51	2.2
17.50	15.00	112.5	0.7	8.75	0.00	8.75	89.85	197.6	3.17	2.77	4.13	4.75	0.57	2.1
20.00	17.50	153.1	3.0	10.45	0.15	10.60	114.05	250.8	3.80	3.35	4.42	5.00	0.60	2.1
22.50	20.00	200.0	7.6	12.12	0.40	12.52	142.95	314.4	4.36	4.06	4.91	5.16	0.62	2.1
25.00	22.50	253.1	14.8	13.80	0.75	14.55	176.75	388.8	5.13	4.88	5.24	5.34	0.63	2.1
27.50	25.00	312.5	25.2	15.50	1.20	16.70	215.75	474.6	5.90	5.85	5.64	5.40	0.62	2.1
30.00	27.50	378.1	39.5	17.19	1.70	18.89	260.25	572.7	6.79	6.84	6.00	5.53	0.62	2.2
32.50	30.00	450.0	58.3	18.88	2.35	21.23	310.40	683.0	7.77	7.92	6.28	5.67	0.61	2.3
35.00	32.50	528.1	81.8	20.57	3.10	23.67	366.52	806.5	8.93	9.13	6.49	5.74	0.60	2.4
37.50	35.00	612.5	111.2	22.26	4.00	26.26	428.92	943.8	10.13	10.48	6.73	5.75	0.59	2.5

The Specific Gravity of the Masonry = 2.2.

TABLE VII.

(See page 6.)

M. CRUGNOLA'S PROFILE-TYPE.

Depth of Water, in Metres.	Length of Joint, in Metres.	Weight per Lineal Metre, in Tons of 2205 lbs.*	Distance of the Line of Pressure from the Centre of the Joint.		Maxima Pressures per Square Centimetre.		Vertical Component	Horizontal Component	Coefficient of Friction necessary for Equilibrium.
			Reservoir Full, in Metres.	Reservoir Empty, in Metres.	At Front Face, in Kilos.	At Back Face, in Kilos.	Of the Resultant of all the Forces in Tons of 2205 lbs.*		
10	6.00	155.82	0.00	0.00	0.000	0.000	155.8	50.0	0.321
15	8.58	239.66	1.00	1.25	5.832	5.232	239.6	112.5	0.469
20	12.52	360.98	1.47	2.20	6.482	5.922	364.4	200.0	0.549
25	16.72	529.11	1.85	2.91	7.186	6.468	543.1	312.5	0.575
30	21.20	747.15	2.05	3.42	7.821	6.936	781.1	450.0	0.576
35	26.96	1018.32	1.95	4.20	7.601	7.309	1083.3	612.5	0.565
40	33.28	1364.70	1.70	4.80	7.480	7.648	1476.7	800 0	0.542
45	39.93	1785.66	1 50	5.20	7.664	7 965	1968.1	1012.5	0.514
50	46.92	2285.05	1.10	5.45	7.721	8.265	2565.0	1250.0	0 487

* 1 Ton = 1 cubic metre of water = 2204.737 lbs. The Specific Gravity of the Masonry = 2.3.

TABLE VIII.

(See page 24.)

THEORETICAL PROFILE No. 1.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.0	0	0	18.74	0.00	18.74*	0.0	9.37	9.37	0.0	0.00	0.0	0.00	0.00	0.0
37.1	295	3648	18.74	0.00	18.74	695.3	4.12	9.37	112.3	8.19†	37.1	2.71	0.42	1.8
50.0	535	8929	24.76	0.00	24.76	975.8	5.79	9.82	112.3	8.19	63.9	4.66	0.55	1.6
60.0	771	15429	30.47	0.00	30.47	1252.0	7.44	10.71	112.3	8.19	78.1	5.69	0.62	1.6
70.0	1049	24500	36.87	0.00	36.87	1588.7	9.43	12.02	112.3	8.19	88.1	6.42	0.66	1.6
80.0	1370	36571	43.87	0.00	43.87	1992.4	11.84	13.68	112.3	8.19	97.2	7.09	0.68	1.6
90.0	1734	52071	51.39	0.00	51.39	2468.7	14.65	15.65	112.3	8.19	105.0	7.66	0.70	1.7
100.0	2141	71429	59.44	0.00	59.44	3022.8	17.94	17.85	112.3	8.19	112.8	8.22	0.71	1.8
110.0	2591	95071	68.02	0.00	68.02	3660.1	21.73	20.32	112.3	8.19	122.2	8.91	0.71	1.8
120.0	3084	123429	77.15	0.00	77.15	4386.0	26.04	22.97	112.3	8.19	127.3	9.28	0.70	1.9
130.0	3619	156929	86.73	0.00	86.73	5205.4	30.77	25.81	112.3	8.19	134.4	9.80	0.69	2.0
140 0	4197	196000	96.72	0.00	96.72	6122.6	35.89	28.82	112.3	8.19	140.4	10.24‡	0.68	2.1
150.0	4818	241071	107.25	2.02	109.27	7152.6	41.61	33.96	112.3	8.19	140.4	10.24	0.67	2.2
160.0	5482	292571	118.21	4.34	122.55	8311.7	47.88	39.47	112.3	8.19	140.4	10.24	0.66	2.4

* The top width was made equal to that of Rankine's Type (see Table III.), for comparison.

† Equivalent to 8 kilos. per square centimetre. ‡ Equivalent to 10 kilos. per square centimetre.

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

TABLE IX.

(See page 24.)

THEORETICAL PROFILE No. 2.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.00	0	0	16.40	0.00	16.40*	0.0	8.20	8.20	0.00	0.00	0.0	0.00	0.00	0.00
37.47	301	3758	16.40	0.00	16.40	614.5	2.09	8.20	196.53	14.34†	37.5	2.73	0.49	1.34
50.00	535	8929	21.95	0.00	21.95	854.8	2.90	8.61	196.53	14.34	64.1	4.67	0.63	1.28
60.00	771	15429	27.19	0.00	27.19	1100.5	3.73	9.44	196.53	14.34	77.7	5.67	0.70	1.26
70.00	1049	24500	32.89	0.00	32.89	1400.9	4.74	10.65	196.53	14.34	87.2	6.36	0.75	1.27
80.00	1370	36571	38.90	0.00	38.90	1759.8	5.98	12.14	196.53	14.34	96.7	7.05	0.78	1.29
90.00	1734	52071	45.14	0.00	45.14	2180.0	7.39	13.86	196.53	14.34	104.9	7.65	0.79	1.31
100.00	2141	71429	51.59	0.00	51.59	2663.7	9.03	15.74	196.53	14.34	112.9	8.23	0.80	1.33
110.00	2591	95071	58.24	0.00	58.24	3212.8	10.90	17.75	196 53	14.34	120.7	8.80	0.80	1.37
120.00	3084	123429	65.10	0.00	65.10	3829.5	13.04	19.84	196.53	14.34	128.7	9.38	0.80	1.41
130.00	3619	156929	72.11	0.00	72.11	4515.6	15.32	22.04	196.53	14.34	136.6	9.96	0.80	1.44
140.00	4197	196000	79.37	0.00	79.37	5273.0	17.88	24.32	196.53	14.34	144.5	10.54	0.80	1.48
150.00	4818	241071	86.87	0.00	86.87	6104.2	20.71	26.67	196.53	14.34	152.6	11.13	0.79	1.53
160.00	5482	292571	94.61	0.00	94.61	7011.6	23.78	29.10	196.53	14.34	160.6	11.71	0.78	1.57

* Equal to 5 metres. † Equivalent to 14 kilos. per square centimetre.

The Specific Gravity of the Masonry = $2\frac{1}{8}$.

TABLE X.

(See page 25.)

THEORETICAL PROFILE No. 3.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.0	0	0	18.74	0.00	18.74*	0	9.37	9.37	0.0	0.00	0.0	0.00	0.00	0.0
26.5	175	1551	18.74	0.00	18.74	497	6.25	9.37	53.0	3.31	26.5	1.66	0.35	3.0
40.0	400	5333	25.08	0.00	25.08	792	8.36	9.99	63.1	3.94	50.8	3.18	0 51	2.2
50.0	625	10417	31.17	0.00	31.17	1074	10.39	11.08	68.9	4.31	64.4	4.03	0.58	2.1
60.0	900	18000	37.92	0.00	37.92	1419	12.64	12.60	74.8	4.68	75.1	4.69	0.63	2.0
70.0	1225	28583	45.52	1.04	46.56	1842	15.52	15.52	79.1	4.94	79.1	4.94	0.66	2.0
80.0	1600	42667	52.83	1.64	54.47	2347	18.16	18.16	86.2	5.39	86.2	5.39	0.68	2.0
90.0	2025	60750	60.16	2.04	62.20	2930	20.73	20.73	94.2	5.89	94.2	5.89	0.69	2.0
100 0	2500	83333	67.36	2.29	69.65	3589	23.22	23.22	103.1	6.44	103.1	6.44	0.69	2.0
110.0	3025	110917	74.52	2.46	76.98	4322	25.66	25.66	112.3	7.02	112.3	7.02	0.70	2.0
120.0	3600	144000	81.65	2.58	84.23	5129	28.08	28.08	121.8	7.61	121.8	7.61	0.70	2.0
130.0	4225	183083	88.77	2.66	91.43	6007	30.48	30.48	131.1	8.19†	131.1	8.19	0.70	2.0
140.0	4900	228667	98.43	3.87	102.30	6976	35.42	34.10	131.1	8.19	136.4	8.53	0.70	2.1
150.0	5625	281250	108.45	5.00	113.45	8054	40.71	37.82	131.1	8.19	145.0	9.06	0.70	2.2
160.0	6400	341333	118.89	6.10	124.99	9246	46.41	41.66	131.1	8.19	148.0	9.25	0.69	2.3
170.0	7225	409417	129.71	7.17	136.88	10556	52.46	45.63	131.1	8.19	154.2	9.64	0.69	2.4
180.0	8100	486000	140.91	8.23	149.14	11986	58.88	49.71	131.1	8.19	160.7	10.04	0.68	2.5
190.0	9025	571583	152.67	10.49	163.16	13547	65.85	55.12	131.1	8.19	163.8	10.24‡	0.67	2.6
200.0	10000	666667	164.97	14.22	179.19	15259	73.88	61.63	131.1	8.19	163.8	10.24	0.65	2.7

* The top width was made equal to that of Rankine's Type (see Table III.), for comparison.

* Equivalent to 8 kilos. per square centimetre. † Equivalent to 10 kilos. per square centimetre.

The Specific Gravity of the Masonry = 2.

TABLE XI.

(See page 25.)

THEORETICAL PROFILE No. 4.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis. Left, in Feet.	Right, in Feet.	Total, in Feet.	Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures. Reservoir Full. In Feet of Masonry.	In Tons of 2000 lbs.	Reservoir Empty. In Feet of Masonry.	In Tons of 2000 lbs.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
0.00	0	0	18.74	0.00	* 18.74	0	9.37	9.37	0.0	0.00	0.0	0.00	0.00	0.0
27.58	176	1614	18.74	0.00	18.74	517	6.25	9.37	55.3	3.74	27.7	1.88	0.34	3.0
40.00	370	4923	24.20	0.00	24.20	784	8.07	9.84	64.8	4.39	50.5	3.42	0.47	2.3
50.00	577	9615	29.87	0.00	29.87	1054	9.96	10.79	70.6	4.78	65.0	4.40	0.55	2.1
60.00	831	16615	36.25	0.00	36.25	1385	12.08	12.18	76.4	5.17	75.6	5.12	0.60	2.0
70 00	1132	26386	43.41	0.87	44.28	1788	14.76	14.76	80.7	5.46	80.7	5.46	0.63	2.0
80.00	1478	39385	50.52	1.55	52.07	2269	17.36	17.36	87.2	5.90	87.2	5.90	0.65	2.0
90.00	1871	56077	57.52	1.99	59.51	2827	19.84	19.84	95.0	6.43	95.0	6.43	0.66	2.0
100.00	2309	76923	64.45	2.27	66.72	3458	22.24	22.24	103.7	7.02	103.7	7.02	0.66	2.0
110.00	2795	102385	71.35	2.47	73.82	4161	24.61	24.61	112.7	7.63	112.7	7.63	0.67	2.0
120.00	3326	132923	78.21	2.61	80.82	4934	26.94	26.94	121.0	8.19†	122.1	8.26	0.67	2.0
130.00	3903	169000	87.68	3.87	91.55	5796	31.84	30.52	121.0	8.19	126.6	8.57	0.67	2.1
140.00	4527	210769	97.30	4.94	102.24	6765	37.00	34.08	121.0	8.19	132.3	8.96	0.67	2.2
150.00	5196	259615	107.44	6.02	113.46	7844	42.54	37.82	121.0	8.19	138.3	9.36	0.66	2.3
160.00	5912	315077	117.94	7.05	124.99	9036	48.46	41.66	121.0	8.19	144.6	9.79	0.65	2.4
170 00	6674	377692	128.81	8.05	136.86	10345	54.73	45.62	121.0	8.19	151.2	10.24‡	0.64	2.5
180 00	7483	448615	140.47	11.30	151.77	11788	61.77	51.94	121.0	8.19	151.2	10.24	0.63	2.6
190.00	8337	527615	152.60	15.09	167.69	13386	69.44	58.83	121.0	8.19	151.2	10.24	0.62	2.7
200.00	9238	615385	165.24	19.47	184.71	15148	77.72	66.36	121.0	8.19	151.2	10.24	0.61	2.9

* The top width was made equal to that of Rankine's Type (see Table III.), for comparison.

† Equivalent to 8 kilos. per square centimetre. ‡ Equivalent to 10 kilos. per square centimetre.

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

TABLE XII.

(See page 25.)

THEORETICAL PROFILE NO. 5.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis. Left, in Feet.	Right, in Feet.	Total, in Feet.	Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures. Reservoir Full. In Feet of Masonry.	In Tons of 2000 lbs.	Reservoir Empty. In Feet of Masonry.	In Tons of 2000 lbs.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
0.00	0	0	18.74	0.00	* 18.74	0	9.37	9.37	0.0	0.00	0.0	0.00	0.00	0.0
28.62	175	1671	18.74	0.00	18.74	536	6.25	9.37	57.2	4.17	28.6	2.09	0.33	3.0
40.00	343	4571	23.45	0.00	23.45	776	7.82	9.74	66.2	4.83	49.7	3.62	0.44	2.3
50.00	535	8929	28.78	0.00	28.78	1038	9.59	10.58	72.2	5.26	65.0	4.74	0.52	2.1
60.00	771	15429	34.83	0.00	34.83	1356	11.61	11.83	77.8	5.67	76.2	5.56	0.57	2.0
70.00	1049	24500	41.37	0.85	42.22	1741	14.07	14.07	82.5	6.01	82.5	6.01	0.60	2.0
80.00	1370	36571	48.28	1.60	49.88	2206	16.63	16.63	88.4	6.45	88.4	6.45	0.62	2.0
90.00	1734	52071	54.97	2.04	57.01	2740	19.00	19.00	96.2	7.01	96.2	7.01	0.63	2.0
100.00	2141	71429	61.68	2.38	64.06	3345	21.35	21.35	104.4	7.61	104.4	7.61	0.64	2.0
110.00	2591	95071	68.34	2.60	70.94	4020	23.65	23.65	112.3	8.19†	112.3	8.19	0.64	2.0
120.00	3084	123429	77.39	3.82	81.21	4781	28.32	27.07	112.3	8.19	117.8	8.59	0.64	2.1
130.00	3619	156929	86.68	4.88	91.56	5645	33.24	30.52	112.3	8.19	123.3	8.99	0.64	2.2
140.00	4197	196000	96.43	5.90	102.33	6615	38.59	34.11	112.3	8.19	129.3	9.43	0.63	2.3
150.00	4818	241071	106.60	6.90	113.50	7694	44.34	37.83	112.3	8.19	135.6	9.89	0.63	2.4
160.00	5482	292571	117.15	7.87	125.02	8886	50.43	41.67	112.3	8.19	140.4	10.24 ‡	0.62	2.5
170.00	6188	350929	128.57	11.67	140.24	10213	57.45	48.43	112.3	8.19	140.4	10.24	0.61	2.6
180.00	6938	416571	140.50	15.46	155.96	11694	65.05	55.28	112.3	8.19	140.4	10.24	0.59	2.8
190.00	7730	489929	152.91	19.95	172.86	13338	73.26	62.87	112.3	8.19	140.4	10.24	0.58	3.0
200.00	8565	571429	165.96	25.02	190.98	15157	82.29	70.99	112.3	8.19	140.4	10.24	0.56	3.2

* The top width was made equal to that of Rankine's Type (see Table III.), for comparison.

† Equivalent to 8 kilos. per square centimetre. ‡ Equivalent to 10 kilos. per square centimetre.

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

TABLE XIII.

(See page 25.)

THEORETICAL PROFILE NO. 6.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.0	0	0	18.74	0.00	18.74	0	9.37	9.37	0.0	0.00	0.0	0.00	0.00	0.0
29.6	175	1729	18.74	0.00	18.74	555	6.25	9.37	59.2	4.63	29.6	2.31	0.32	3.0
40.0	320	4267	22.80	0.00	22.80	771	7.60	9.66	67.6	5.28	49.3	3.85	0.41	2.4
50.0	500	8333	27.85	0.00	27.85	1024	9.28	10.42	73.5	5.74	64.6	5.04	0.49	2.2
60.0	720	14400	33.58	0.00	33.58	1331	11.19	11.57	79.3	6.19	76.6	5.99	0.54	2.0
70.0	980	22867	39.91	0.39	40.30	1701	13.43	13.43	84.4	6.59	84.4	6.59	0.58	2.0
80.0	1280	34133	46.62	1.20	47.82	2141	15.94	15.94	89.5	6.99	89.5	6.99	0.59	2.0
90.0	1620	48600	53.20	1.72	54.92	2655	18.31	18.31	96.7	7.55	96.7	7.55	0.61	2.0
100.0	2000	66667	59.69	2.07	61.76	3238	20.59	20.59	104.9	8.19*	104.9	8.19	0.62	2.0
110.0	2420	88733	68.08	3.16	71.24	3903	24.76	23.75	104.9	8.19	109.6	8.56	0.62	2.1
120.0	2880	115200	78.97	4.20	81.17	4665	29.42	27.06	104.9	8.19	115.0	8.98	0.62	2.2
130.0	3380	146467	86.32	5.21	91.53	5529	34.53	30.51	104.9	8.19	120.8	9.44	0.61	2.3
140.0	3920	182933	96.10	6.19	102.29	6498	40.03	34.10	104.9	8.19	127.1	9.93	0.60	2.4
150.0	4500	225000	106.40	7.89	114.29	7581	46.06	38.55	104.9	8.19	131.1	10.24 †	0.59	2.6
160.0	5120	273066	117.39	11.15	128.54	8796	52.84	44.65	104.9	8.19	131.1	10.24	0.58	2.7
170.0	5780	327533	128.93	15.00	143.93	10158	60.30	51.39	104.9	8.19	131.1	10.24	0.57	2.9
180.0	6480	388800	141.06	19.49	160.55	11680	68.45	58.81	104.9	8.19	131.1	10.24	0.56	3.1
190.0	7220	457286	153.82	24.66	178.48	13375	77.35	66.94	104.9	8.19	131.1	10.24	0.54	3.3
200.0	8000	533333	167.25	30.57	197.82	15257	87.03	75.83	104.9	8.19	131.1	10.24	0.52	3.5

The Specific Gravity of the Masonry = $2\frac{1}{2}$.

* Equivalent to 8 kilos. per square centimetre.

† Equivalent to 10 kilos. per square centimetre.

TABLE XIV.

(See page 27.)

INCLINED JOINTS IN THEORETICAL PROFILE No. 5.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry.	Moment of Water, in Cubic Feet of Masonry.	Joint. Below Top of Dam. Front Edge.	Joint. Below Top of Dam. Back Edge.	Joint. Length.	Angle with the Horizon.	Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Maxima Pressures, Reservoir Full. In Feet of Masonry.	Maxima Pressures, Reservoir Full. In Tons of 2000 lbs.	Coefficient of Friction necessary for Equilibrium.
28.62	175	1671	18.62	28.62	21.23	28° 05′	442.64	9.55	28.97	2.11	0.11
28.62	175	1671	23.62	28.62	19.38	14° 56′	489.49	7.59	44.21	3.22	0.08
28.62	175	1671	28.62	28.62	18.74	0	536.34	6.25	57.20	4.17	0.33
28.62	175	1671	33.62	28.62	21.37	13° 32′	583 20	7.49	46.75	3.41	0.58
28.62	175	1671	40.00	28.62	26.04	25° 53′	642.96	9.00	32.42	2.36	0.87
60.00	771	15429	45.00	60.00	30.12	29° 53′	1096.18	10.48	84.72	6.18	0.10
60.00	771	15429	47.50	60.00	30.19	24° 29′	1138.79	10.27	88.04	6.42	0.17
60.00	771	15429	50.00	60.00	30.50	19° 10′	1181.36	10.17	89.77	6.55	0.25
60.00	771	15429	52.50	60.00	31.12	13° 55′	1224.89	10.31	88.94	6.49	0.33
60.00	771	15429	55.00	60.00	32.20	8° 56′	1268.45	10.68	85.76	6.26	0.41
60.00	771	15429	60.00	60.00	34.83	0	1355.70	11.61	77.80	5.67	0.57
60.00	771	15429	65.00	60.00	38.46	7° 29′	1437.59	13.14	67.18	4.90	0.72
60.00	771	15429	70.00	60.00	42.55	13° 35′	1519.67	15.08	57.10	4.17	0.85
60.00	771	15429	75.00	60.00	47.31	18° 30′	1605.80	17.44	48.37	3.53	0.97
110.00	2591	95071	85.00	110.00	59.67	24° 45′	3133.69	20.78	125.80	9.18	0.40
110.00	2591	95071	87.50	110.00	60.16	21° 56′	3222.52	20.68	127.66	9.32	0.30
110.00	2591	95071	90.00	110.00	60.98	19° 10′	3311.35	20.85	126.99	9.27	0.34
110.00	2591	95071	95.00	110.00	62.76	13° 50′	3488.53	21.16	126.37	9.22	0.42
110.00	2591	95071	100.00	110.00	64.98	8° 51′	3665.71	21.69	123.83	9.04	0.49
110.00	2591	95071	110.00	110.00	70.94	0	4020.40	23.65	113.40	8.28	0.64
110.00	2591	95071	115.00	110.00	75.63	3° 48′	4197.76	26.25	101.93	7.44	0.71
110.00	2591	95071	120.00	110.00	80.64	7° 08′	4375.12	28.91	92.32	6.74	0.77
110.00	2591	95071	125.00	110.00	86.70	9° 59′	4553.71	32.68	80.98	5.91	0.83
160.00	5482	292571	110.00	160.00	91.08	33° 16′	5815.18	32.99	157.75	11.51	0.18
160.00	5482	292571	115.00	160.00	92.40	29° 08′	6118.39	33.35	158.93	11.60	0.23
160.00	5482	292571	120.00	160.00	94.12	25° 08′	6421.60	33.96	158.68	11.58	0.27
160.00	5482	292571	125.00	160.00	96.42	21° 16′	6727.60	34.93	156.68	11.44	0.32
160.00	5482	292571	130.00	160.00	99.33	17° 36′	7033.60	36.30	152.21	11.11	0.37
160.00	5482	292571	135.00	160.00	102.46	14° 07′	7337.11	37.77	147.58	10.77	0.42
160.00	5482	292571	140.00	160.00	105.82	10° 52′	7640.62	39.50	142.10	10.37	0.46
160.00	5482	292571	145.00	160.00	110.29	7° 48′	7950.97	41.95	134.55	9.82	0.50
160.00	5482	292571	150.00	160.00	114.94	5° 00′	8261.32	44.55	126.81	9.26	0.54
160.00	5482	292571	155.00	160.00	119.85	2° 24′	8573.86	47.39	119.50	8.72	0.58
160.00	5482	292571	160.00	160.00	125.02	0	8886.30	50.43	112.30	8.20	0.62
160.00	5482	292571	165.00	160.00	131.58	2° 11′	9198.94	54.60	103.10	7.52	0.65
160.00	5482	292571	170.00	160.00	137.00	4° 12′	9511.48	58.03	96.80	7.06	0.68
160.00	5482	292571	175.00	160.00	142.86	6° 01′	9823.18	61.85	90.09	6.57	0.71
160.00	5482	292571	200.00	160.00	178.57	12° 58′	11337.30	86.01	61.19	4.47	0.80

TABLE XV.

(See page 27.)

THEORETICAL PROFILE NO. 5, MODIFIED BY BOUVIER'S FORMULÆ.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis. Left, in Feet.	Joint Referred to a Vertical Axis. Right, in Feet.	Joint Referred to a Vertical Axis. Total, in Feet.	Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures. Reservoir Full. In Feet of Masonry.	Maxima Pressures. Reservoir Full. In Tons of 2000 lbs.	Maxima Pressures. Reservoir Empty. In Feet of Masonry.	Maxima Pressures. Reservoir Empty. In Tons of 2000 lbs.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
100	2141	71429	62.75	2.84	65.59	3353.00	22.43	21.86	140.26	10.23	102.24	7.46	0.64	2.05
110	2591	95071	71.71	3.95	75.66	4058.25	27.00	25.24	140.31	10.24	107.28	7.83	0.64	2.15
120	3084	123429	81.04	4.95	85.99	4866.50	31.95	28.68	140.43	10.25	113.19	8.26	0.63	2.20
130	3619	156929	90.76	5.88	96.64	5779.65	37.31	32.23	140.10	10.22	119.61	8.73	0.63	2.38
140	4197	196000	100.79	6.75	107.54	6800.55	42.86	35.86	140.50	10.25	126.48	9.23	0.62	2.50
150	4818	241071	111.18	7.59	118.77	7932.10	48.78	39.60	140.48	10.25	133.57	9.75	0.61	2.60
160	5482	292571	121.89	8.56	130.45	9178.20	54.98	43.59	140.51	10.25	140.44	10.25	0.60	2.72
170	6188	350929	133.45	11.95	145.40	10557 45	62.09	50.07	140.36	10.24	140.43	10.25	0.59	2.87
180	6938	416571	145.36	15.94	161.30	12090 95	69.65	57.19	140.55	10.26	140.33	10.24	0.57	3.02
190	7730	489929	157.74	20.53	178.27	13788.80	77.83	64.91	140.39	10.25	140.47	10.25	0.56	3.19
200	8565	571429	170.56	25.79	196.35	15661.90	86.57	73.30	140.32	10.24	140.39	10.25	0.55	3.37

TABLE XVI.

(See page 32.)

THEORETICAL TYPE No. I.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry.	Moment of Water in Cubic Feet of Masonry.	Length of Joint, in Feet.	Total Area, in Square Feet.	Distance from Centre of Joint to Line of Pressure (Reservoir Full or Empty), in Feet.	Maxima Pressures, Reservoir Full or Empty. In Feet of Masonry.	Maxima Pressures, Reservoir Full or Empty. In Tons of 2000 lbs.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
0	0	0.0	0.00	00.0	0.00	0	0.00	0.000	0
10	21	71.4	6.55	32.7	1.09	10	0.73	0.655	2
20	84	571.4	13.09	130.9	2.18	20	1.46	0.655	2
30	193	1928.0	19.64	294.6	3.27	30	2.19	0.655	2
40	343	4571.4	26.19	523.7	4.36	40	2.92	0.655	2
50	535	8928.6	32.73	818.3	5.45	50	3.65	0.655	2
60	771	15428.6	39.28	1178.4	6.55	60	4.38	0.655	2
70	1049	24500.0	45.83	1603.9	7.64	70	5.11	0.655	2
80	1370	36571.4	52.37	2094 9	8.73	80	5.84	0.655	2
90	1734	52071.4	58.92	2651.4	9.82	90	6.57	0.655	2
100	2141	71428.6	65.47	3273.3	10.91	100	7.30	0.655	2
110	2591	94071.4	72.01	3960.7	12.00	110	8.03	0.655	2
120	3084	123428.6	78.56	4713.5	13.09	120	8.76	0.655	2
130	3619	156928.6	85.11	5531.9	14.18	130	9.49	0.655	2
140	4197	196000.0	91.65	6415.6	15.27	140	10.22	0.655	2
150	4818	241071.4	98.20	7364.9	16.37	150	10.95	0.655	2
160	5482	292571.4	104.75	8379.7	17.46	160	11.68	0.655	2
170	6188	350928.6	111.29	9459.8	18.55	170	12.41	0.655	2
180	6938	416571.4	117.84	10605.5	19.64	180	13.14	0.655	2
190	7730	489928.6	124.39	11816.6	20.73	190	13.87	0 655	2
200	8565	571428.6	130.93	13093.2	21.82	200	14.60	0.655	2

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

TABLE XVII.

(See page 34.)

PRACTICAL TYPE No. I.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis. Left, in Feet.	Right, in Feet.	Total, in Feet.	Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures. Reservoir Full. In Feet of Masonry.	Reservoir Full. In Tons of 2000 lbs.	Reservoir Empty. In Feet of Masonry.	Reservoir Empty. In Tons of 2000 lbs.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
0.00	0	0.0	20.00	0.00	20.00	0.0	10.00	10.00	0.0	0.00	0.0	0.00	0.00	0.0
10.00	21	71.4	20.00	0.00	20.00	200.0	9.64	10.00	11.1	0.81	10.0	0.73	0.11	27.8
20.00	84	571.4	20.00	0.00	20.00	400.0	8.57	10.00	28.6	2.09	20.0	1.46	0.21	7.0
30.55	200	2036.6	20.00	0.00	20.00	611.0	6.67	10.00	61.1	4.46	30.6	2.23	0.33	2.0
40.00	343	4571.4	26.19	0.00	26.19	829.2	10.25	10.42	52.3	3.82	51.0	3.72	0.41	2 9
50.00	535	8928.6	32.73	0.00	32.73	1123.8	13.61	11.57	51.6	3.77	64.5	4.71	0.47	2.8
60.00	771	15428.6	39.28	0.00	39.28	1483.9	15.74	13.14	60.2	4.39	75.2	5.49	0.52	2.5
70.00	1049	24500.0	45.83	0.00	45.83	1909.4	18.04	14.96	68.2	4.98	85.1	6.21	0.55	2.4
80.00	1370	36571.4	52.37	0.00	52.37	2400.4	20.21	16.93	77.3	5.64	94.5	6.90	0.57	2.3
90.00	1734	52071.4	58.92	0.00	58.92	2956.9	22.32	18.99	86.6	6.32	103.8	7.58	0.59	2.3
100 00	2141	71428.6	65.47	0.00	65.47	3578.8	24.41	21.09	96.4	7.04	113.1	8.25	0.60	2.2
110.00	2591	94071.4	72.01	0.00	72.01	4266 2	26.48	23.24	106.3	7.76	122.4	8.94	0.61	2.2
120.00	3084	123428.6	78.56	0.00	78.56	5019.1	28.57	25.40	116.1	8.48	131.7	9.61	0.61	2.1
130.00	3619	156928.6	85.11	0.00	85.11	5837.4	30.65	27.58	126.2	9.21	141.1	10.30	0.62	2.1
140.00	4197	196000.0	91.65	0.00	91.65	6721.2	32.72	29.77	136.2	9.94	150.5	10.98	0.62	2.1
150.00	4818	241071.4	98.20	0.00	98.20	7670.4	34.72	31.96	146.8	10.72	160.0	11.68	0.63	2.1
160.00	5482	292571.4	104.75	0.00	104.75	8685.1	36.90	34.16	156.3	11.41	169.5	12.37	0.63	2.1
170.00	6188	350928.6	111.29	0.00	111.29	9765.3	39.00	36.35	166.5	12.15	179.1	13.07	0.63	2.1
180.00	6938	416571.4	117.84	0.00	117.84	10911.0	41.10	38.55	176.7	12.90	188.7	13.76	0.63	2.1
190.00	7730	489928.6	124.39	0.00	124.39	12122.1	43.22	40.75	186.7	13.63	198.3	14.46	0.64	2.1
200.00	8565	571428.6	130.93	0.00	130.93	13398.7	45.33	42.95	196.6	14.35	207.9	15.16	0.64	2.1

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

NOTE.—The Profile given by this Table can be changed to another having any desired top width (equal to $\frac{1}{10}$ the height) by simply changing the scale to which it has been drawn. To obtain a corresponding Table from the one above, proceed as follows: Let r = ratio of desired top width to that of Practical Type No. I. Divide the numbers in columns 1, 4, 5, 6, 8, 9, 10, 11, 12, 13, by r; those in column 2 and 7 by r^2; those in column 3 by r^3. The numbers in column 14 and 15 will remain unchanged.

TABLE XVIII.

(See page 34.)

THEORETICAL TYPE No. II.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
			Left, in Feet.	Right, in Feet.	Total, in Feet.				Reservoir Full. In Feet of Masonry.	Reservoir Full. In Tons of 2000 lbs.	Reservoir Empty. In Feet of Masonry.	Reservoir Empty. In Tons of 2000 lbs.		
0.00	0	0.0	20.00	0.00	20.00	0.0	10.00	10.00	0.0	0.00	0.0	0.00	0.00	0.0
30.55	200	2036.6	20.00	0.00	20.00	611.0	6.67	10.00	61.1	4.45	30.5	2.23	0.33	2.4
40.00	343	4571.4	23.75	0.00	23.75	817.7	7.92	10.24	68.8	5.02	48.6	3.54	0.42	2.2
50.00	535	8928.6	28.83	0.00	28.83	1080.6	9.61	10.96	74.9	5.46	64.4	4.70	0.49	2.1
60.00	771	15428.6	34.68	0.00	34.68	1398.2	11.56	12.09	80.6	5.88	76.9	5.61	0.55	2.0
67.00	962	21483.1	39.08	0.00	39.08	1656.3	13.03	13.08	84.7	6.17	84.6	6.17	0.60	2.0
80.00	1370	36571.4	48.07	1.13	49.20	2230.2	16.40	16.40	90.6	6.60	90.6	6.60	0.61	2.0
90.00	1734	52071.4	54.90	1.72	56.62	2759.3	18.87	18.87	97.4	7.10	97.4	7.10	0.62	2.0
100.00	2141	71428.6	61.63	2.12	63.75	3361.1	21.25	21.25	105.4	7.68	105.4	7.68	0.64	2.0
110.00	2591	95071.4	68.31	2.40	70.71	4033.4	23.57	23.57	114.1	8.32	114.1	8.32	0.64	2.0
120.00	3084	123428.6	74.95	2.60	77.55	4774.7	25.85	25.85	123.1	8.97	123.1	8.97	0.64	2.0
130.00	3619	156928.6	81.57	2.74	84.31	5584.0	28.10	28.10	132.5	9.66	132.5	9.66	0.65	2.0
140.00	4197	196000.0	88.17	2.84	91.01	6460.6	30.34	30.34	142.0	10.35	142.0	10.35	0.65	2.0
150.00	4818	241071.4	94.76	2.92	97.68	7404.1	32.56	32.56	151.6	11.05	151.6	11.05	0.65	2.0
160.00	5482	292571.4	101.33	2.98	104.31	8414.0	34.77	34.77	161.3	11.76	161.3	11.76	0.65	2.0
170.00	6188	350928.6	107.90	3.03	110.93	9490.2	36.98	36.98	171.1	12.47	171.1	12.47	0.65	2.0
180.00	6938	416571.4	114.47	3.07	117.54	10632.6	39.18	39.18	180.9	13.19	180.9	13.19	0.65	2.0
190.00	7730	489928.6	121.03	3.10	124.13	11840.9	41.38	41.38	190.8	13.91	190.8	13.91	0.65	2.0
200.00	8565	571428.6	127.59	3.12	130.71	13115.1	43.57	43.57	200.7	14.63	200.7	14.63	0.65	2.0

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

NOTE.—The Profile given by this Table can be changed to another having any desired top width (equal to $\frac{1}{10}$ the height) by simply changing the scale to which it has been drawn. To obtain a corresponding Table from the one above, proceed as follows: Let r = ratio of desired top width to that of Theoretical Type No. II. Divide the numbers in columns 1, 4, 5, 6, 8, 9, 10, 11, 12, 13 by r; those in columns 2 and 7 by r^2; those in column 3 by r^3. The numbers in columns 14 and 15 will remain unchanged.

TABLE XIX.

(See page 35.)

PRACTICAL TYPE No. 2.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
			Left, in Feet.	Right, in Feet.	Total, in Feet.				Reservoir Full. In Feet of Masonry.	Reservoir Full. In Tons of 2000 lbs.	Reservoir Empty. In Feet of Masonry.	Reservoir Empty. In Tons of 2000 lbs.		
0.000	0	0	20.00	0.00	20.00	0.0	10.00	10.00	0.0	0.00	0.0	0.00	0.00	0.0
18.744	75	470	20.00	0.00	20.00	374.9	8.74	10.00	25.8	1.89	18.7	1.36	0.20	8.0
30.000	192	1929	21.07	0.00	21.07	604.0	7.86	10.06	50.4	3.68	32.5	2.37	0.31	3.4
40.000	343	4571	23.89	0.00	23.89	827.2	7.99	10.37	68.9	5.03	48.3	3.53	0.41	2.4
51.967	579	10024	30.04	0.00	30.04	1146.5	10.07	11.21	75.8	5.53	67.3	4.91	0.50	2.1
60.000	771	15429	35.38	0.00	35.38	1409.1	12.26	12.17	76.6	5.59	77.1	5.63	0.54	2.1
70.000	1049	24500	42.03	0.62	42.65	1799.3	14.71	14.33	81.4	5.94	83.7	6.11	0.58	2.1
80.000	1370	36571	48.68	1.25	49.93	2262.2	17.07	16.70	88.3	6.45	90.3	6.59	0.61	2.1
90.000	1734	52071	55.33	1.87	57.20	2797.9	19.39	19.20	96.1	7.02	97.1	7.09	0.62	2.0
100.000	2141	71429	61.98	2.50	64.48	3406.2	21.73	21.78	104.4	7.62	104.2	7.61	0.63	2.0
110.000	2591	95071	68.63	3.12	71.75	4087.4	24.09	24.40	113.2	8.26	111.7	8.15	0.63	2.0
120.000	3084	123429	75.28	3.74	79.02	4840.7	26.46	27.06	121.9	8.90	119.1	8.69	0.63	2.0
130.000	3619	156929	81.93	3.74	85.67	5664.2	28.85	29.11	130.9	9.55	129.6	9.46	0.64	2.0
140.000	4197	196000	88.58	3.74	92.32	6554.2	31.22	31.20	140.0	10.22	140.0	10.22	0.64	2.0
150.000	4818	241071	95.23	3.74	98.97	7510.7	33.56	33.32	149.2	10.89	150.3	10.96	0.64	2.0
160.000	5482	292571	101.88	3.74	105.62	8533.7	35.87	35.46	158.5	11.56	160.5	11.71	0.64	2.0
170.000	6188	350929	108.53	3.74	112.27	9623.2	38.20	37.61	168.0	12.25	170.5	12.44	0.64	2.0
180.000	6938	416571	115.18	3.74	118.92	10779.2	40.50	39.78	177.5	12.95	180.7	13.18	0.64	2.0
190.000	7730	489929	121.83	3.74	125.57	12001.7	42.80	41.95	186.9	13.63	190.7	13.91	0.64	2.0
200.000	8565	571429	128.48	3.74	132.22	13290.6	45.10	44.13	196.4	14.32	200.8	14.65	0.64	2.0

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

NOTE.—The Profile given by this Table can be changed to another having any desired top width (equal to $\frac{1}{10}$ the height) by simply changing the scale to which it has been drawn. To obtain a corresponding Table from the one above, proceed as follows: Let r = ratio of desired top width to that of Practical Type No. 2. Divide the numbers in columns 1, 4, 5, 6, 8, 9, 10, 11, 12, 13, by r; those in columns 2 and 7 by r^2; those in column 3 by r^3. The numbers in columns 14 and 15 will remain unchanged.

TABLE XX.

(See page 37.)

PRACTICAL PROFILE No. 1.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.000	0	0	5.00	0.00	5.00	0.0	2.50	2.50	0.00	0.00	0.00	0.00	0.00	0.0
4.686	5	7	5.00	0.00	5.00	23.4	2.18	2.50	6.45	0.48	4.70	0.34	0.20	8.0
10.000	22	71	5.98	0.00	5.98	51.7	2.00	2.60	17.25	1.26	12.10	0.89	0.41	2.4
12.992	36	157	7.51	0.00	7.51	71.7	2.52	2.81	18.95	1.39	16.85	1.23	0.50	2.1
15.000	48	241	8.85	0.00	8.85	88.1	3.07	3.05	19.15	1.40	19.30	1.41	0.54	2.1
20.000	86	571	12.17	0.32	12.49	141.4	4.27	4.18	22.10	1.62	22.60	1.65	0.61	2.1
25.000	134	1116	15.50	0.62	16.12	212.9	5.44	5.45	26.10	1.92	26.05	1.90	0.63	2.0
30.000	193	1929	18.82	0.94	19.76	302.6	6.62	6.77	30.50	2.23	29.80	2.18	0.63	2.0
35.000	262	3063	22.14	0.94	23.08	409.7	7.81	7.80	35.00	2.51	35.00	2.55	0.64	2.0
40.000	343	4571	25.47	0.94	26.41	533.4	8.97	8.87	39.65	2.39	40.15	2.93	0.64	2.0
45.000	434	6509	28.80	0.94	29.74	673.7	10.13	9.95	44.40	3.24	45.20	3.30	0.64	2.0
50.000	535	8929	32.12	0.94	33.06	830.7	11.28	11.04	49.10	3.58	50.20	3.67	0.64	2.0

The Specific Gravity of the Masonry = 2⅓.

TABLE XXI.

(See page 37.)

PRACTICAL PROFILE NO. 2.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir Full.		Reservoir Empty.			
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.000	0	0	10.00	0.00	10.00	0.0	5.00	5.00	0.0	0.00	0.0	0.00	0.00	0.0
9.372	19	59	10.00	0.00	10.00	93.7	4.37	5.00	12.9	0.95	9.4	0.68	0.20	8.0
15.000	48	241	10.54	0.00	10.54	151.0	3.93	5.03	25.2	1.84	16.3	1.69	0.31	3.4
20.000	86	571	11.95	0.00	11.95	206.8	4.00	5.18	34.5	2.52	24.2	1.77	0.41	2.4
25.983	145	1253	15.02	0.00	15.02	286.6	5.04	5.60	37.9	2.77	33.7	2.45	0.50	2.1
30.000	193	1929	17.69	0.00	17.69	352.3	6.13	6.08	38.3	2.80	38.6	2.82	0.54	2.1
35.000	262	3063	21.02	0.31	21.33	449.8	7.36	7.16	40.7	2.97	41.9	3.06	0.58	2.1
40.000	343	4571	24.34	0.63	24.97	565.6	8.54	8.35	44.2	3.23	45.2	3.30	0.61	2.1
45.000	434	6509	27.66	0.94	28.60	699.5	9.70	9.60	48.1	3.51	48.6	3.55	0.62	2.0
50.000	535	8929	30.99	1.25	32.24	851.6	10.87	10.89	52.2	3.81	52.1	3.81	0.63	2.0
55.000	648	11884	34.32	1.56	35.88	1021.9	12.05	12.20	56.6	4.13	55.9	4.08	0.63	2.0
60.000	771	15429	37.64	1.87	39.51	1210.2	13.23	13.53	61.0	4.45	59.6	4.35	0.63	2.0
65.000	905	19616	40.97	1.87	42.84	1416.1	14.43	14.55	65.5	4.78	64.8	4.73	0.64	2.0
70.000	1049	24500	44.29	1.87	46.16	1638.6	15.61	15.60	70.0	5.11	70.0	5.11	0.64	2.0
75.000	1205	30134	47.62	1.87	49.49	1877.7	16.78	16.66	74.6	5.45	75.2	5.48	0.64	2.0
80.000	1371	36571	50.94	1.87	52.81	2133.4	17.94	17.73	79.3	5.78	80.3	5.86	0.64	2.0
85.000	1547	43866	54.27	1.87	56.14	2405.8	19.10	18.80	84.0	6.13	85.3	6.22	0.64	2.0
90.000	1735	52071	57.59	1.87	59.46	2694.8	20.25	19.89	88.8	6.48	90.4	6.59	0.64	2.0
95.000	1933	61241	60.92	1.87	62.79	3000.4	21.40	20.98	93.5	6.82	95.4	6.96	0.64	2.0
100.000	2141	71429	64.24	1.87	66.11	3322.7	22.55	22.07	98.2	7.16	100.4	7.33	0.64	2.0

The Specific Gravity of the Masonry = 2⅓.

TABLE XXII.

(See page 37.)

PRACTICAL PROFILE NO. 3.

Depth of Water Below Top of Dam, in Feet.	Horizontal Thrust of Water, in Cubic Feet of Masonry	Moment of Water, in Cubic Feet of Masonry.	Joint Referred to a Vertical Axis.			Total Area, in Square Feet.	Distance from Front Face to Line of Pressure, Reservoir Full, in Feet.	Distance from Back Face to Line of Pressure, Reservoir Empty, in Feet.	Maxima Pressures. Reservoir Full.		Maxima Pressures. Reservoir Empty.		Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
			Left, in Feet.	Right, in Feet.	Total, in Feet.				In Feet of Masonry.	In Tons of 2000 lbs.	In Feet of Masonry.	In Tons of 2000 lbs.		
0.000	0	0.0	18.74	0.00	18.74	0.00	9.37	9.37	0.00	0.00	0.00	0.00	0.00	0.0
16.585	59	325.9	18.74	0.00	18.74	310.80	8.32	9.37	22.16	1.62	16.59	1.21	0.19	8.9
20.000	86	571.4	18.86	0.00	18.86	374.98	7.93	9.41	29.37	2.14	20.01	1.46	0.23	6.2
30.000	193	1928.5	20.56	0.00	20.56	570.33	7.67	9.51	48.87	3.57	33.97	2.48	0.34	3.3
40.000	343	4571.4	24.52	0.00	24.52	793.65	8.78	9.98	59.93	4.38	50.43	3.68	0.43	2.5
50.000	535	8928.6	29.95	0.00	29.95	1065.69	10.65	10.92	66.41	4.84	64.49	4.70	0.50	2.2
60.000	771	15428.6	35.71	0.43	36.14	1395.80	12.44	12.64	74.72	5.45	73.44	5.35	0.55	2.1
70.000	1049	24500.0	41.81	0.87	42.68	1789.57	14.40	14.59	82.84	6.04	81.72	5.96	0.59	2.0
80.000	1370	36571.4	48.29	1.30	49.59	2250.59	16.62	16.72	90.28	6.58	89.73	6.54	0.61	2.0
90.000	1734	52071.4	55.15	1.73	56.88	2782.60	19.16	19.01	96.81	7.06	97.58	7.12	0.62	2.0
100.000	2141	71428.6	62.41	2.17	64.58	3389.58	22.06	21.45	102.38	7.46	105.35	7.68	0.63	2.0
110.000	2591	95071.4	70.11	2.60	72.71	4075.87	25.37	24.02	106.87	7.79	113.12	8.25	0.63	2.1
120.000	3084	123428.6	78.28	3.65	81 93	4848.70	29 16	27.32	110.34	8.05	118.31	8.63	0.63	2.1
130.000	3619	156928.6	86.94	4.71	91.65	5716 17	33.45	30.75	112.90	8.23	123.92	9.04	0.63	2.2
140.000	4197	196000.0	96.13	5.76	101.89	6683.36	38.28	34.28	114.51	8.35	129.96	9.48	0.63	2.3
150.000	4818	241071.4	105.90	6.82	112.72	7755.93	43.69	37.95	115.21	8.40	136.23	9.94	0.62	2.4
160.000	5482	292571.4	116.32	7.87	124.19	8939.87	49.72	41.75	115.02	8.39	142.74	10.48	0.61	2.5
170.000	6188	350928.6	127.44	12.16	139.60	10258.14	56.51	48.88	115.45	8.42	139.54	10.18	0.60	2.6
180.000	6938	416571.4	139.34	16.45	155.79	11734.32	64.24	56.05	114.93	8.38	138.69	10.11	0.59	2.8
190.000	7730	489928.6	152.14	20.73	172.87	13376.80	72.98	63 27	113.52	8.28	139.59	10.18	0.58	3.0
200.000	8565	571428.6	165.96	25.02	190.98	15195.10	82.75	70.62	111.40	8.13	141.72	10.33	0.56	3.2

The Specific Gravity of the Masonry = $2\frac{1}{3}$.

TABLE XXIII.*
DATA OF MASONRY DAMS.

No.	Dam.	Location.	Date of Construction.	Depth of Water, in Feet.	Total Height above Bed-rock, in Feet.	Thickness at Top, in Feet.	Thickness at Base, in Feet.	Area of Profile, in Sq. Feet.	Maximum Pressure in the Masonry, in Tons of 2000 lbs. per Sq. Ft.	Weight of Masonry, per Cub. Ft.	Length at Top, in Feet	Length at Base, in Feet	Plan.	Descript'n, page
1	Almanza	Spain	Prior to 1586	......	67.88	9.84	33.73	1496	14.33	....	292	45	Curved $R = 86.07$	43
2	Elche	Spain	About 1570—1590	64.64	76.12	29.52	39.37	2615	13.00	....	230	60	Curved $R = 205.33$	46
3	Alicante	Spain	1579—1594	127.14	134.52	65.62	110.57	11839	11.54	....	190	30	Curved $R = 351.37$	43
4	Lampy	France	1776—1782	51.35	52.17	16.08	36.65	1229	....	....	....	....	....	52
5	Puentes	Spain	1785—1791	153.54	164.24	35.73	144.29	16349	8.12	....	925	70	Polygonal	46
6	Val de Infierno	Spain	1785—1791	......	116.48	41.14	136.98	11668	6.66	....	330	67	Polygonal	48
7	Gros-Bois	France	1830—1838	71.52	92.90	21.33	45.93	2503	14.59	....	1805	....	Straight	52
8	Vioreau	France	1833—1838	32.81	36.09	24.61	24.61	872	6.14	....	....	....	Straight	52
9	Bosmelea	France	1833—1838	46.91	50.20	14.11	27.88	987	8.61	....	....	....	Straight	52
10	Glomel	France	1833—1838	39.04	42.94	13.78	24.54	774	9.41	....	....	....	Straight	52
11	Tillot	France	....	......	65.62	17.88	17.88	1173	....	....	....	....	....	53
12	Chazilly	France	....	......	73.80	13.39	53.15	......	....	....	1759	....	....	53
13	Zola	France	About 1843	119.76	123.02	19.03	41.83	3645	8.12	....	205	23	Curved $R = 158$	53
14	Nijar	Spain	1843—1850	82.03	90.33	24.28	67.59	5386	7.68	....	356	....	Curved	49
15	Lozoya	Spain	1852	94.00	105.00	21.98	127.96	9052	....	....	238	200	Straight	49
16	Furens	France	1862—1866	164.04	170.60	9.91	161.02	10712	6.82	....	328	30	Curved $R = 828.38$	53
17	Ternay	France	1865—1868	112.70	124.68	13.12	81.69	4355	7.16	147.0	....	....	Curved $R = 1312.4$	56
18	Habra	Algiers	1865—1873	116.81	124.68	14.11	95.00	5584	6.31	134.0	1476	....	Straight	62
19	Cagliari	Italy	1866	......	70.54	16.40	52.50	2430	....	....	345	164	....	69
20	Verdon	France	1866—1870	41.17	58.00	14.99	32.55	863	5.81	....	131	....	Curved $R = 108.83$	57
21	Boyd's Corner	United States	1866—1872	......	78.00	8.60	57.00	2039	....	....	670	200	Straight	84
22	Ban	France	1867—1870	137.90	156.82	16.40	126.98	6780	8.18	....	....	....	Curved	57
23	Tlelat	Algiers	1869	......	68.90	13.12	40.34	......	6.14	....	325	....	Straight	66
24	Gileppe	Belgium	1869—1875	147.64	154.20	49.22	216.50	18708	6.14	143.5	771	269	Curved $R = 1640.4$	70
25	Villar	Spain	1870—1878	162.30	170.33	14.75	154.50	11596	9.60	143.0	546	....	Curved $R = 440$	50
26	Pas du Riot	France	1872—1878	......	113.19	......	......	......	....	....	....	....	Curved	56
27	Djidionia	Algiers	1873—1875	......	83.67	13.12	52.50	......	9.65	....	196	....	Straight	67
28	Geelong	Australia	....	......	60.00	2.50	44.00	1214	4.00	143.0	226	....	Curved $R = 300$	81
29	Poona	India	....	......	108.00	13.75	60.75	3725	....	150.0	....	....	....	75
30	Toolsee	India	....	......	79.00	18.74	50.35	2527	6.70	....	....	....	....	83
31	Hijar	Spain	1880	......	141.08	16.40	146.98	8454	5.12	....	236	....	Curved $R = 210$	50
32	Gorzente	Italy	1882	121.40	126.32	13.12	99.58	5657	....	....	492	....	....	69
33	Bouzey	France	1882	......	75.46	13.12	45.93	1961	11.26	....	1545	....	Straight	58
34	Gran Cheurfas	Algiers	1882—1884	......	131.24	13.12	134.52	......	6.14	....	509	164	Straight	67
35	Pont	France	1883	......	85.31	16.40	62.34	2755	....	....	495	....	Curved $R = 1312.4$	59
36	Hamiz	Algiers	1885	114.84	134.52	16.40	91.21	5629	11.25	....	532	131	Straight	68
37	Bridgeport	United States	1886—1888	......	40.00	......	......	......	....	....	640	50	Straight	85
38	Vyrnwy	England	1882—1889	......	146.00	20.00	117.80	8972	8.70	160.8	1350	....	Straight	71
39	Vingeanne	France	1885	......	113.85	11.48	80.12	......	....	....	....	....	....	56
40	Tâche	France	1888—1892	......	161.43	......	......	......	11.00	150.0	....	....	Curved $R = 1312.4$	59
41	Cotatay	France	1885	......	113.19	......	......	......	....	....	....	....	....	56
42	Tytam	China	1888	......	95.00	23.47	62.85	3978	....	....	....	....	....	82
43	San Mateo	United States	1887—1889	......	170.00	20.00	176.00	16660	....	....	700	....	Curved $R = 637$	86
44	Sodom	United States	1888—1893	......	98.00	12.00	53.00	......	....	...	500	150	Straight	92
45	Tansa	India	1886—1891	......	118.00	12.00	100.00	......	9.00	....	8800	....	....	75
46	Bear Valley	United States	1884	60.00	64.00	3.17	20.00	537	....	....	450	....	Curved $R = 300$	87
47	Sweetwater	United States	1886—1888	90.00	98.00	12.00	46.00	2347	....	164.0	380	....	Curved $R = 222$	87

REMARKS.—The Puentes Dam was ruptured on the 30th of April, 1802. The Habra Dam failed in December, 1881.

* This Table has been compiled from the authorities mentioned in our descriptions of dams and from Mr. Crugnola's excellent work on Retaining Walls and Dams.

(*Continued on page* 233.)

TABLE XXIII.—*Continued.*

DATA OF MASONRY DAMS.

No.	Dam.	Location.	Date of Construction.	Depth of Water, in Feet.	Total Height above Bed-rock, in Feet.	Thickness at Top, in Feet.	Thickness at Base, in Feet.	Area of Profile, in Sq. Feet.	Maximum Pressure in the Masonry, in Tons of 2000 lbs. per Sq. Ft.	Weight of Masonry per Cubic Foot in Lbs.	Length at Top, in Feet.	Length at Base, in Feet.	Plan.	Description, page.
48	Bhatgur...........	India................	....	......	130	12	74	......	7.5	....	4067	....	....	77
49	Betwa...............	India	....	......	60	....	.....	....	....	....	3296	....	Curved	78
50	Periar..............	India	1888—1897	153	180	12	136	10772	....	....	1200	215	Straight	79
51	Beetaloo...........	South Australia.....	1888—1890	......	110	14	110	......	....	....	580	....	Curved R=1414	80
52	Mouche............	France.............	Completed 1890	95	101	11.5	.....	......	6.7	134	1346	....	Straight	60
53	Colorado...........	United States.......	Completed 1892	60	67	...	.....	......	....	....	1275	....	Straight	91
54	Butte City..........	United States.......	1892	......	120	10	83	......	... .	....	350	....	Curved R= 350	91
55	Titicus.............	United States.......	1890—1895	......	135	18	75	5209	....	....	534	....	Straight	93
56	Remscheid.........	Germany	1889—1892	......	82	13.12	49.21	......	....	150	....	....	Curved R= 410	74
57	Lagrange...........	United States	1890	......	125	24	90	......	....	....	320	....	Curved R= 300	89
58	Einsiedel...........	Germany	1890—1894	......	92	13.10	65.50	......	....	....	590	....	Curved R=1310	74
59	Hemmet	United States......	1891—1895	......	135.50	10	100	......	....	....	. ..	....	Curved R= 225.40	90
60	Wigwam...........	United States.......	1893—1896	......	75	....	.....	.....	....	....	....	....	Straight	85
61	New Croton	United States	Commenced 1892	150	291	18	200	28400	16.0	....	2240	....	Straight	105

TABLE XXIV.

EQUIVALENTS OF THE METRIC MEASURES, ACCORDING TO THE UNITED STATES STANDARD

1 metre = 39.3685 inches = 3.28071 feet.
1 square metre = 10.763058 square feet.
1 hectare = 107630.58 square feet = 2.47086 acres.
1 litre = 61.0165 cubic inches.
1 cubic metre = 35.3105 cubic feet = 264.141 U. S. liquid gallons.
1 kilogramme = 2.204737 pounds avoirdupois.
1 kilogramme per square centimetre = 1.02421 tons of 2000 pounds per square foot.

Metres.	METRES INTO FEET.									
	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1	3.281	3.609	3.937	4.265	4.593	4.921	5.249	5.577	5.905	6.233
2	6.561	6.889	7.218	7.546	7.874	8.202	8.530	8.858	9.186	9.514
3	9.842	10.170	10.498	10.826	11.154	11.482	11.811	12.139	12.467	12.795
4	13.123	13.451	13.779	14.107	14.435	14.763	15.091	15.419	15.747	16.075
5	16.404	16.732	17.060	17.388	17.716	18.044	18.372	18.700	19.028	19.356
6	19.684	20.012	20.340	20.668	20.996	21.324	21.653	21.981	22.309	22.637
7	22.965	23.293	23.621	23.949	24.277	24.605	24.933	25.261	25.589	25.918
8	26.246	26.574	26.902	27.230	27.558	27.886	28.214	28.542	28.871	29.199
9	29.527	29.855	30.183	30.511	30.840	31.168	31.496	31.824	32.152	32.480
10	32.807	33.135	33.463	33.791	34.119	34.447	34.775	35.103	35.432	35.760

This Table can be used for smaller or larger numbers by changing the position of the decimal-point.

CALCULATION OF THEORETICAL PROFILE No. 6 (TABLE XIII, PAGE 226) BY THE EQUATIONS GIVEN IN PART I, CHAPTER III.

ASSUMED DATA.

Top-width of profile	18.74 feet
Specific gravity of masonry	$2\frac{1}{2}$
Limiting pressure at front face	16,380 lbs. per square foot
Limiting pressure at back face	20,480 lbs. per square foot
Unit of weight	1 cubic foot of masonry

Surface of water assumed at top of dam.

The top-width and limiting pressures are the same as those assumed by Professor Rankine for his profile type (Plate III, Table III). The limiting pressures at the front and back face are respectively equal to 8 and 10 kilogrammes per square centimetre.

We first calculate to what depth both faces may remain vertical by formula (1) on page 17. Substituting the proper values for the known quantities, we obtain

$$d = 18.74\sqrt{\frac{5}{2}} = 29.60.$$

We check this result by calculating v by formula (G), page 16,

$$v = \frac{1729}{555} = 3.12.$$

As $m = 9.37$, n will be $6.25 = \frac{a}{3}$, which proves our calculation to be correct.

We calculate next the length of the joint at a depth of 40 feet by equation (2), page 18. Substituting the values for the known quantities, we get

$$x^2 + \left(\frac{4 \times 555}{10.4} + 18.74\right)x = \frac{6}{10.4}(555 \times 9.37 + 4267) + 351.19;$$

whence $x = 22.80.$

We now check our calculation by moments, assuming a vertical axis of moments 50 feet (any convenient distance) up-stream from the back face of the dam.

Weights.		Lever-arms.		Moments.
555	×	59.37	=	32950
195	×	59.37	=	11577
21	×	70.09	=	1472
771	×	59.66	=	45999

The lever-arm 59.66 is obtained by dividing the sum of the moments by the sum of the weights.

From formula (G), page 16, we find $v = 6.54$. Knowing m and v we find $n = 6.60 = \frac{x}{3}$,

which proves the correctness of our calculations. It is convenient to make a sketch (Fig. 74), showing how the joint calculated is divided by P and P'.

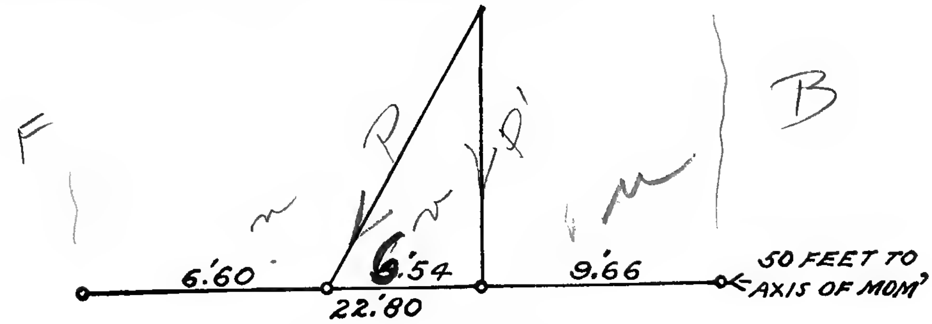


FIG. 74.

660
554
466

We now calculate the length of the joint at a depth of 50 feet by equation (2), page 18. Substituting the values of the known quantities, we obtain

$$x^2 + \left(\frac{4 \times 771}{10} + 22.8\right)x = \frac{6}{10}(771 \times 9.66 + 8333) + 519.84;$$

whence $$x = 27.82.$$

In checking this calculation by moments, we carry forward the total weight and total moment of the previous calculation and add the weight and moment of the course just determined, which is divided into a rectangle and triangle for convenience.

Weights.	Lever-arms.	Moments.
771	9.66	45999
228	61.40	13999
25	74.47	1862
1024	60.41	61860

From formula (G), page 16, we obtain

$$v = 8.14.$$

The joint will be divided by P and P' as follows:

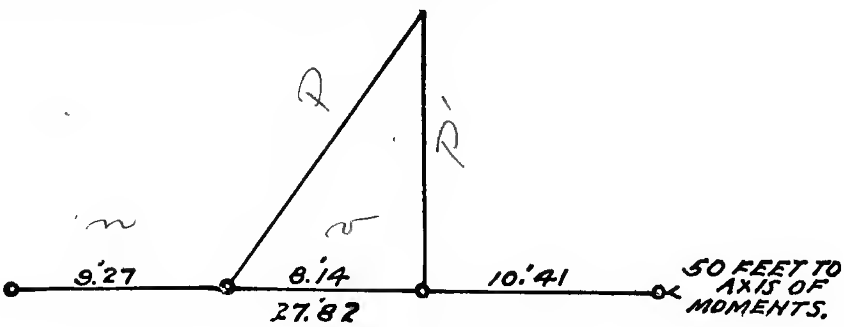


FIG. 75.

The calculations for the first three courses of the dam suffice to show the writer's method of determining the length of a joint and of verifying it by moments. The use of the different equations which must be applied successively in determining the lengths of the joints are explained in Part I, Chapter III.

The Assuan Dam.—A masonry dam is being constructed by the Egyptian Government across the Nile at Assuan, about 500 miles above Cairo, to form a large reservoir for irrigation. This reservoir will be about 130 miles long and will store about 1,065 million cubic metres (37,605 million cubic feet). It will be formed by building a masonry dam having a length of about 1,950 metres (6,397 feet), including the overflow-weir and a lock on the west bank. The dam will be founded entirely on rock and will have a maximum height of about 90 feet above the foundation. Sluiceways, 2 by 7 metres in section, are to be constructed in the dam 7 metres apart, between centres. Plate LXXVIIIA shows two cross-sections of this dam.*

The dam is being built principally of stone quarried from ledges of syenitic granite near the site of the works. It is the same stone of which the obelisks in Central Park, New York, on London's Thames Embankment, and in the Place de la Concorde in Paris, are composed.

The plans for the dam and reservoir were prepared by Mr. W. Willcocks, an Anglo-Indian engineer in the service of the Egyptian Government. Sir Benjamin Baker of London acted as Consulting Engineer. The plans were approved in 1894. A considerable amount of preliminary work was accomplished in 1897. The contract for the dam and reservoir was given in February, 1898, to English contractors, who agreed to complete the work by July 1st, 1903. No payments are to be made for the work until it has been completed. The contractors are then to receive annually $800,000 for a period of thirty years, aggregating $24,000,000. The present value of these payments is not much over $10,000,000.

Besides the Assuan dam, another masonry dam is to be built across the Nile at Assiout (about 200 miles above Cairo). This dam is to direct the river into a system of irrigating canals. It will have a maximum height of about 48 feet. It will consist of a masonry-dam 844 metres long extended on both sides by earth-banks; the total length of the structure including a lock on the west bank being about 1,200 metres. As provision has to be made for passing the whole river Nile for several months each year, the dam is to have in its masonry structure 120 openings, each five metres wide, which are to be controlled by gates. The piers between these openings are generally two metres wide, but every thirtieth pier is given twice this width.

We have been unable to find any detailed technical description of the Nile dams and reservoirs. The information given above has been obtained from the following sources:

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Public Works in Egypt, in "The Architect and Contract Reporter," of London, Supplement of Nov. 11th, 1898;

Harnessing the Nile, by F. C. Penfield, "The Century Magazine," for Feb., 1899.

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* These cross-sections are taken from a copy of the official drawings, signed by Sir William E. Garstin, Under-Secretary of Public Works, which were kindly loaned to the author by Mr. Frederick Cope Whitehouse.

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SAZILLY'S PROFILE TYPE

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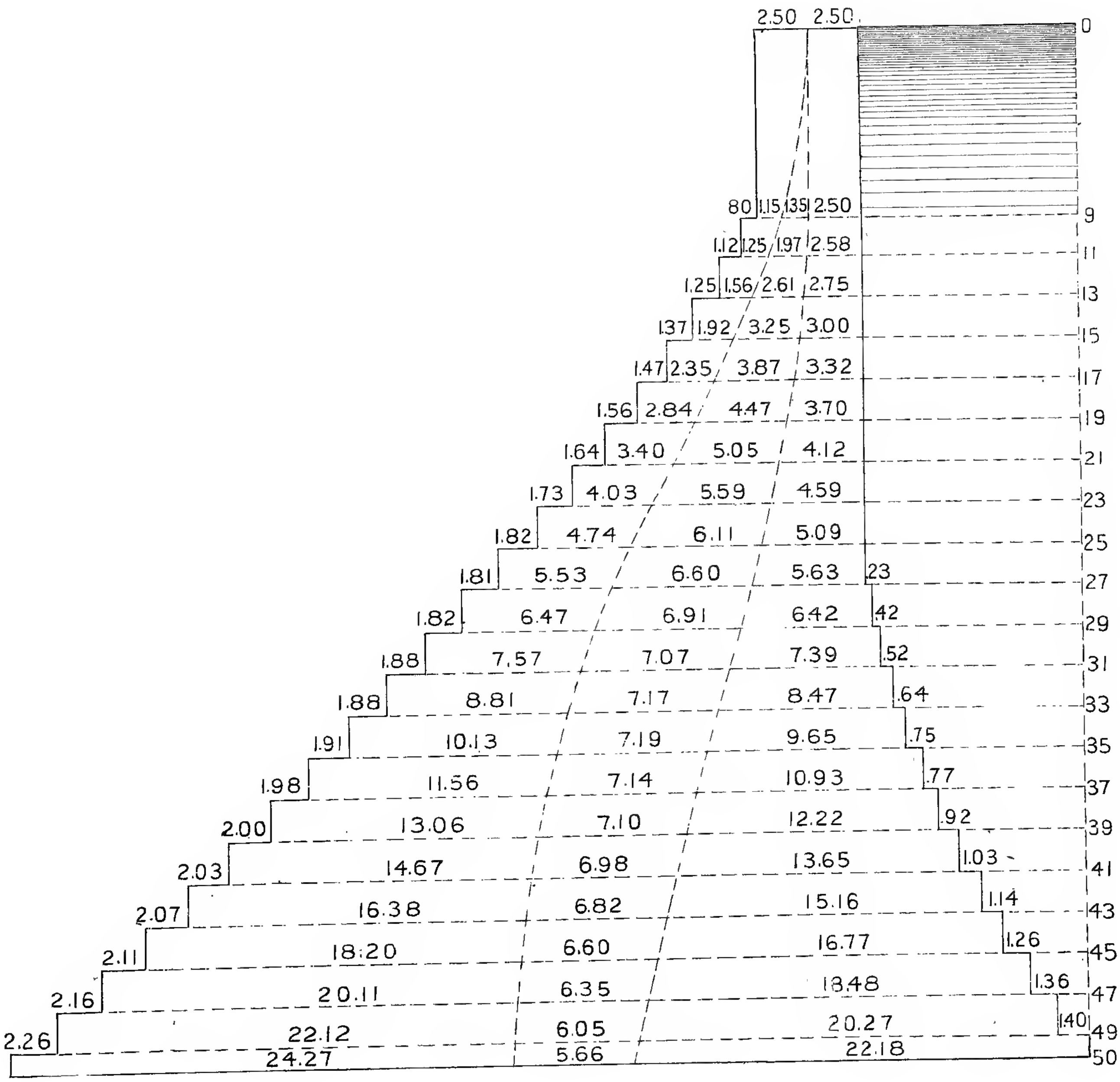

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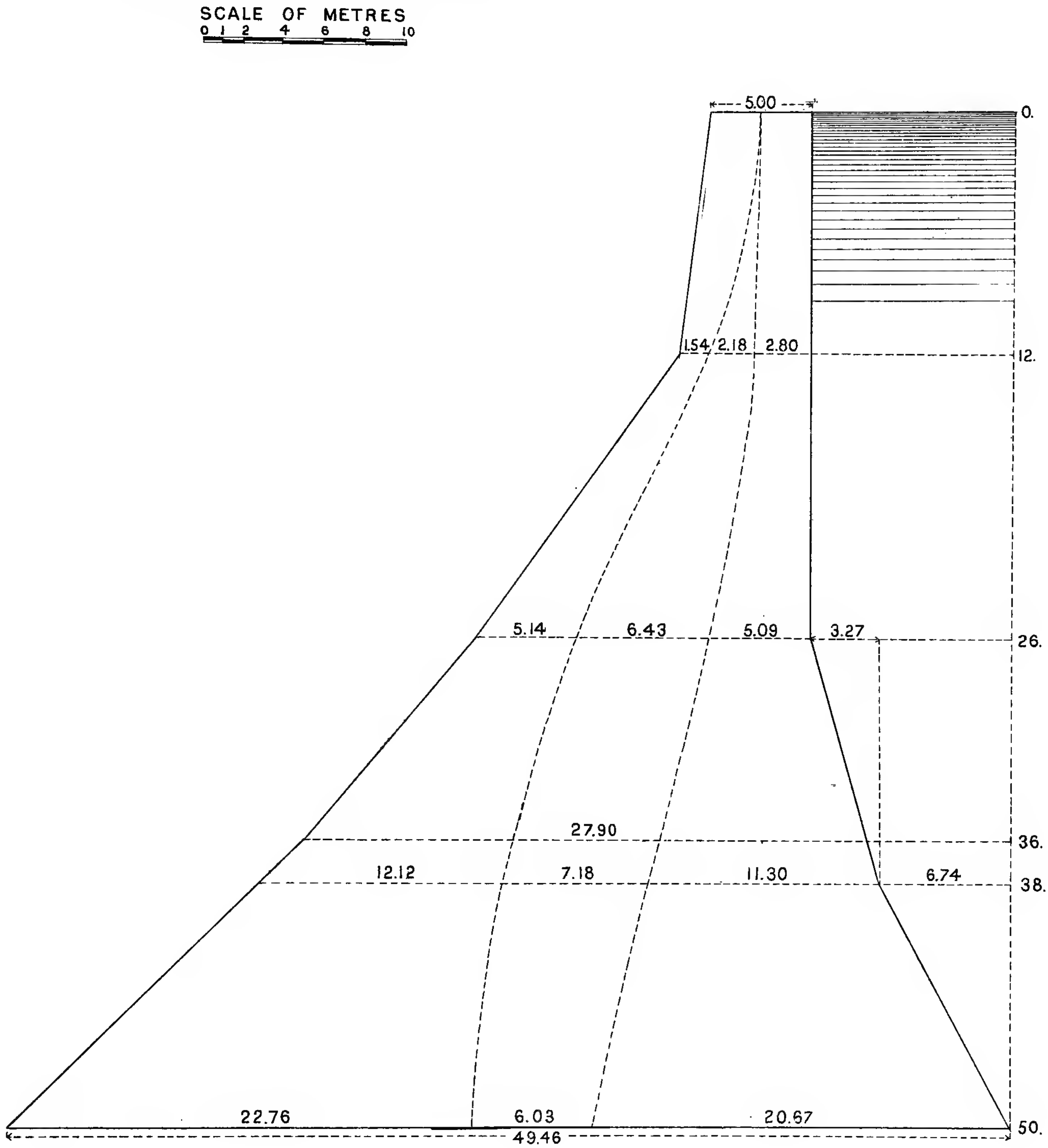

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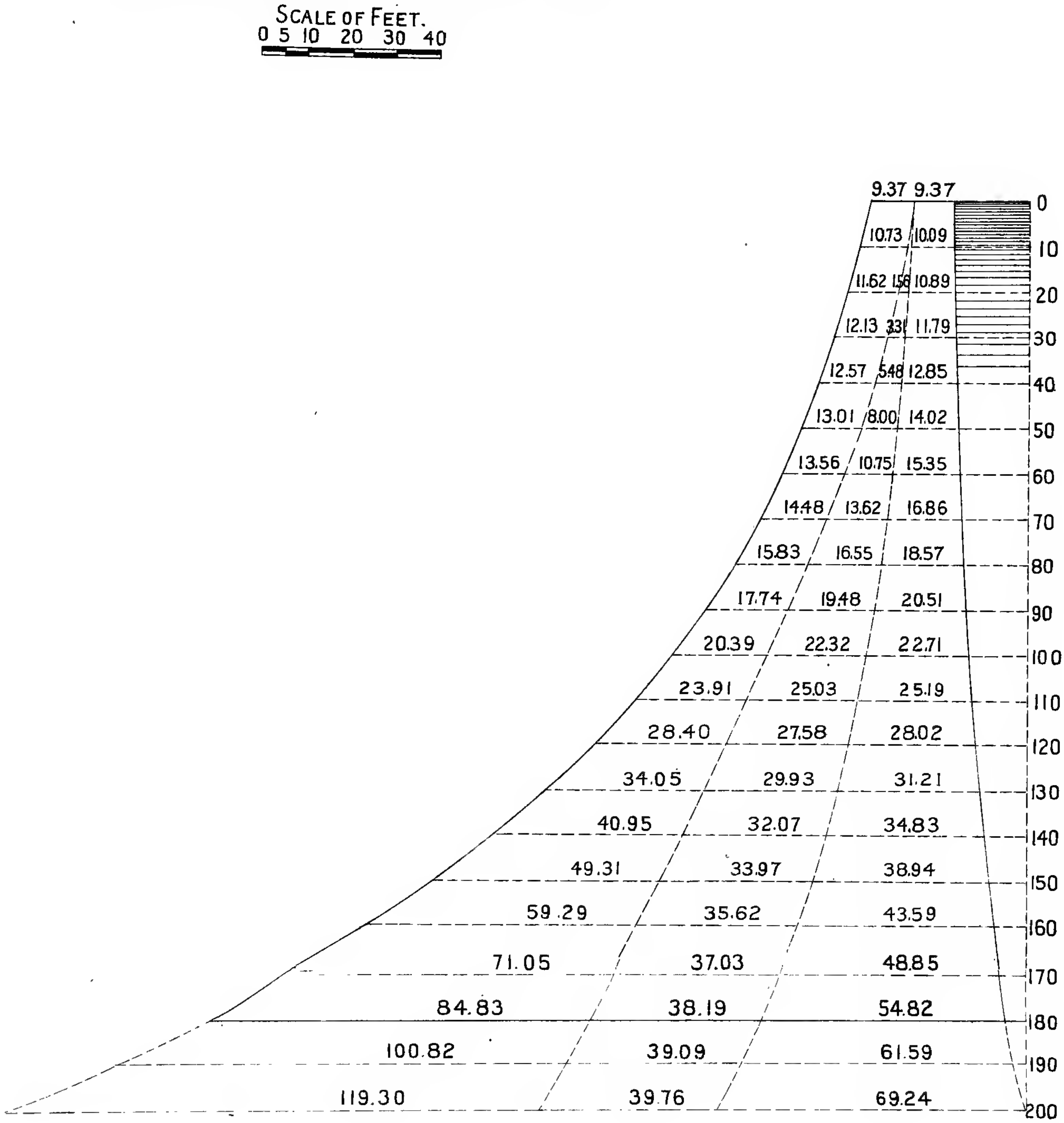

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COMPARISON OF RANKINE'S PROFILE

WITH THE

THEORETICAL PROFILE

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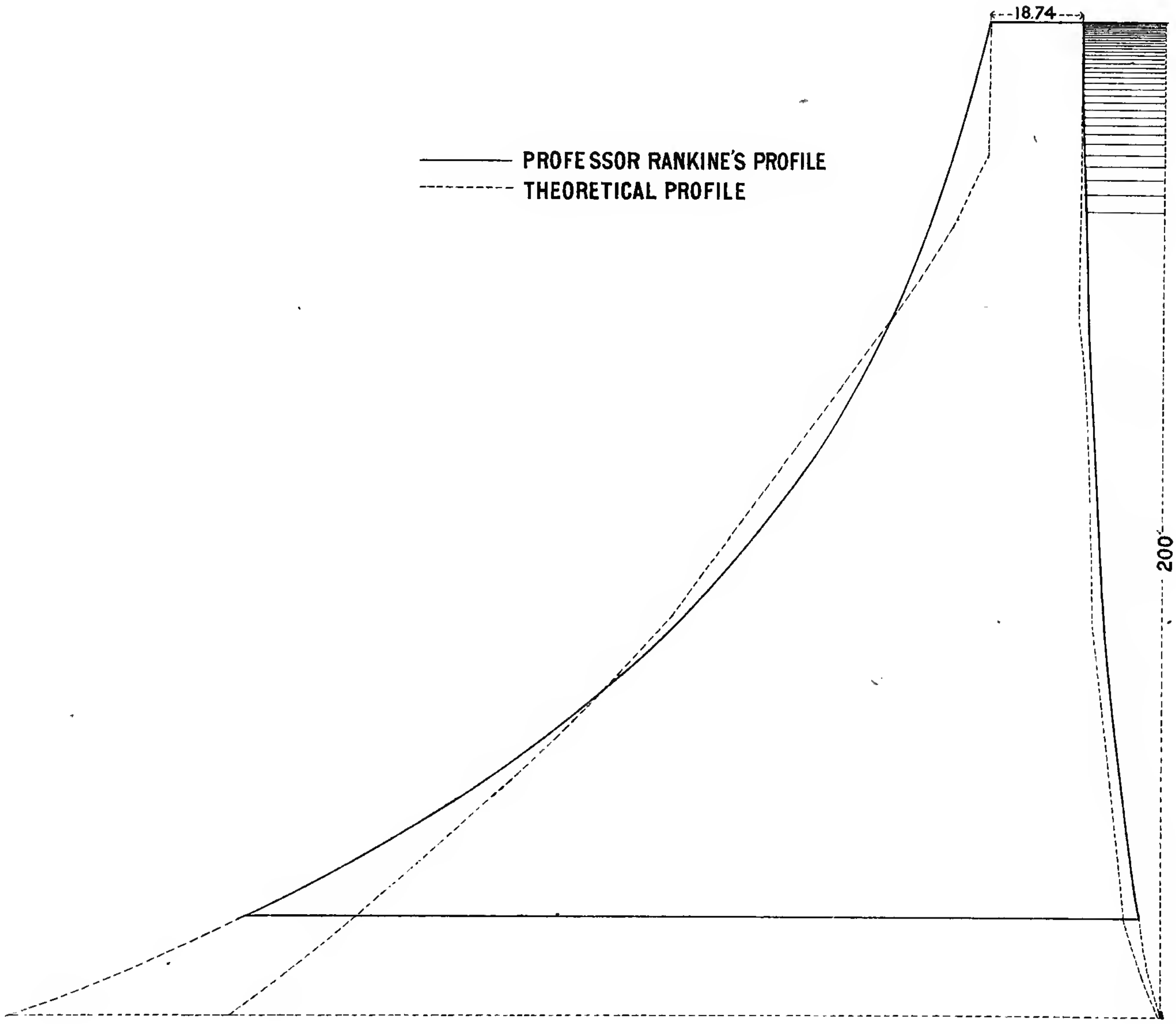

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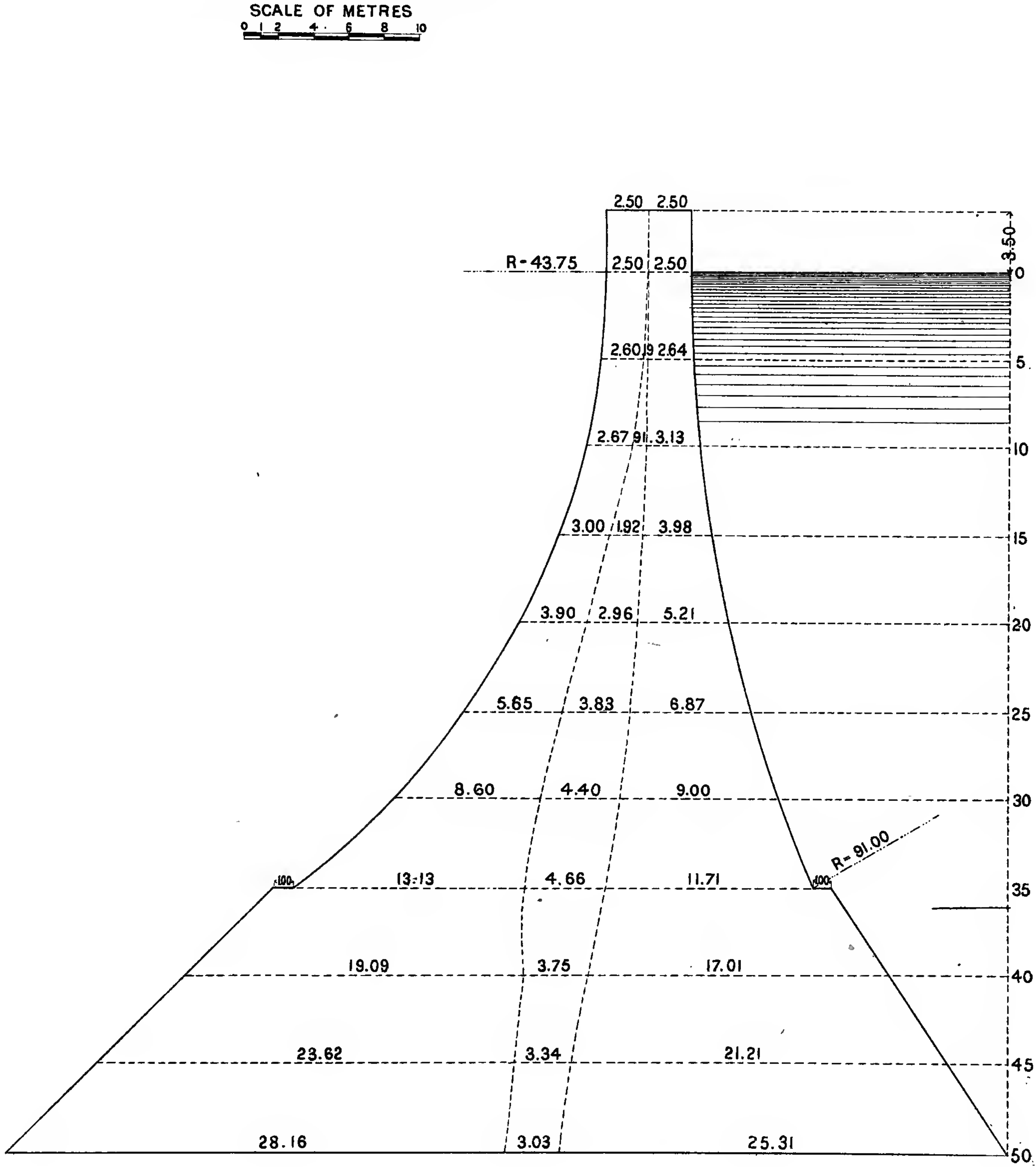

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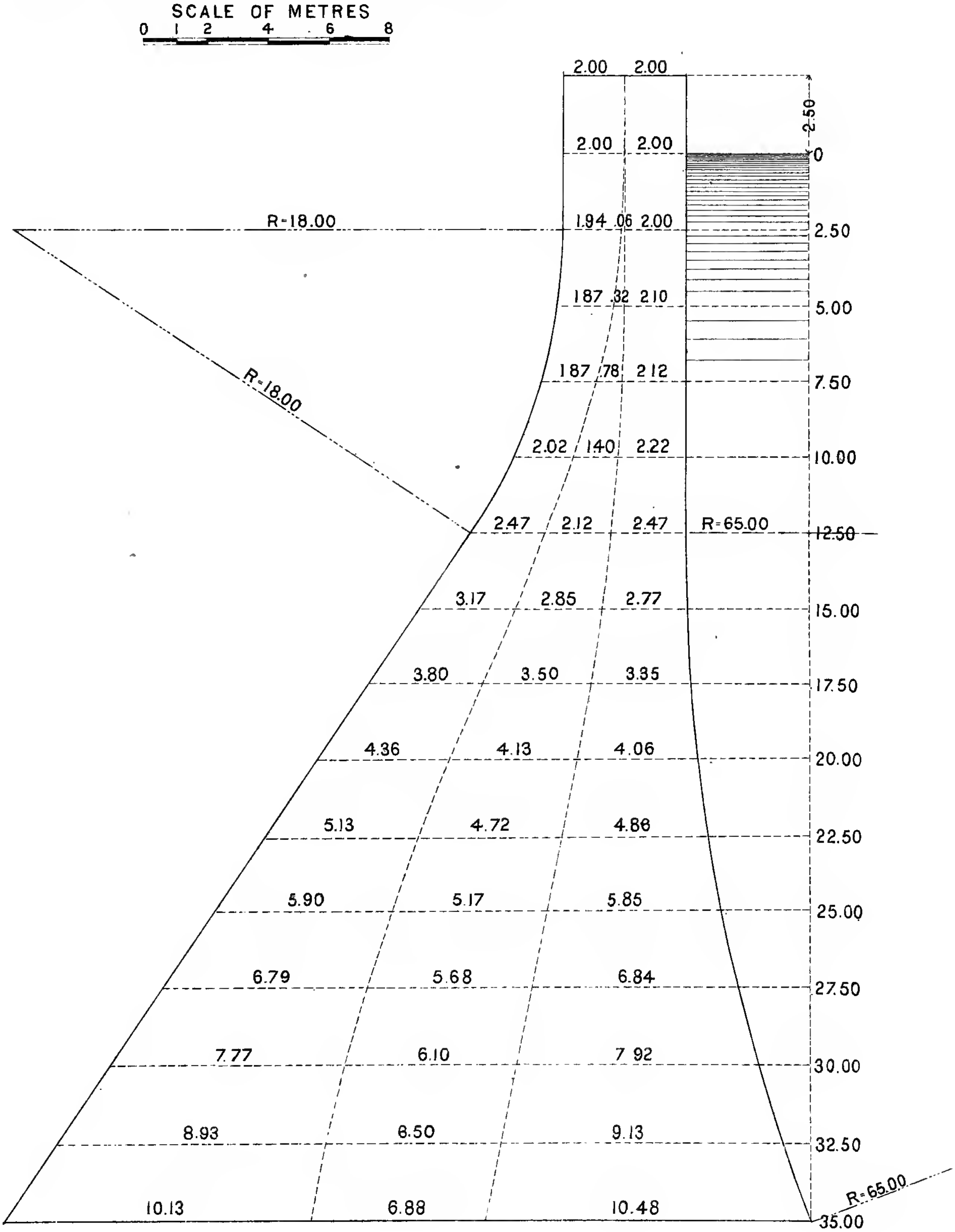

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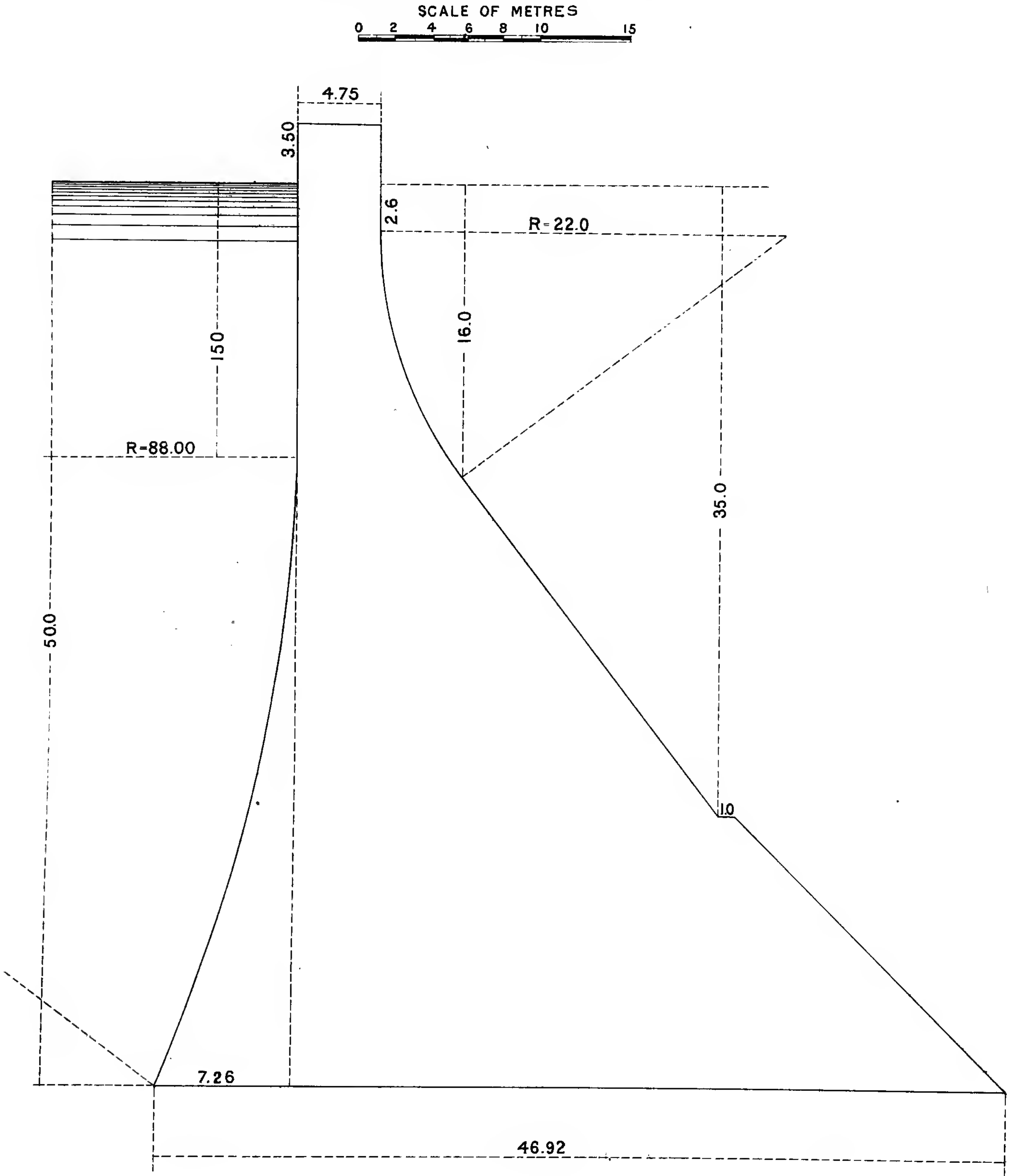

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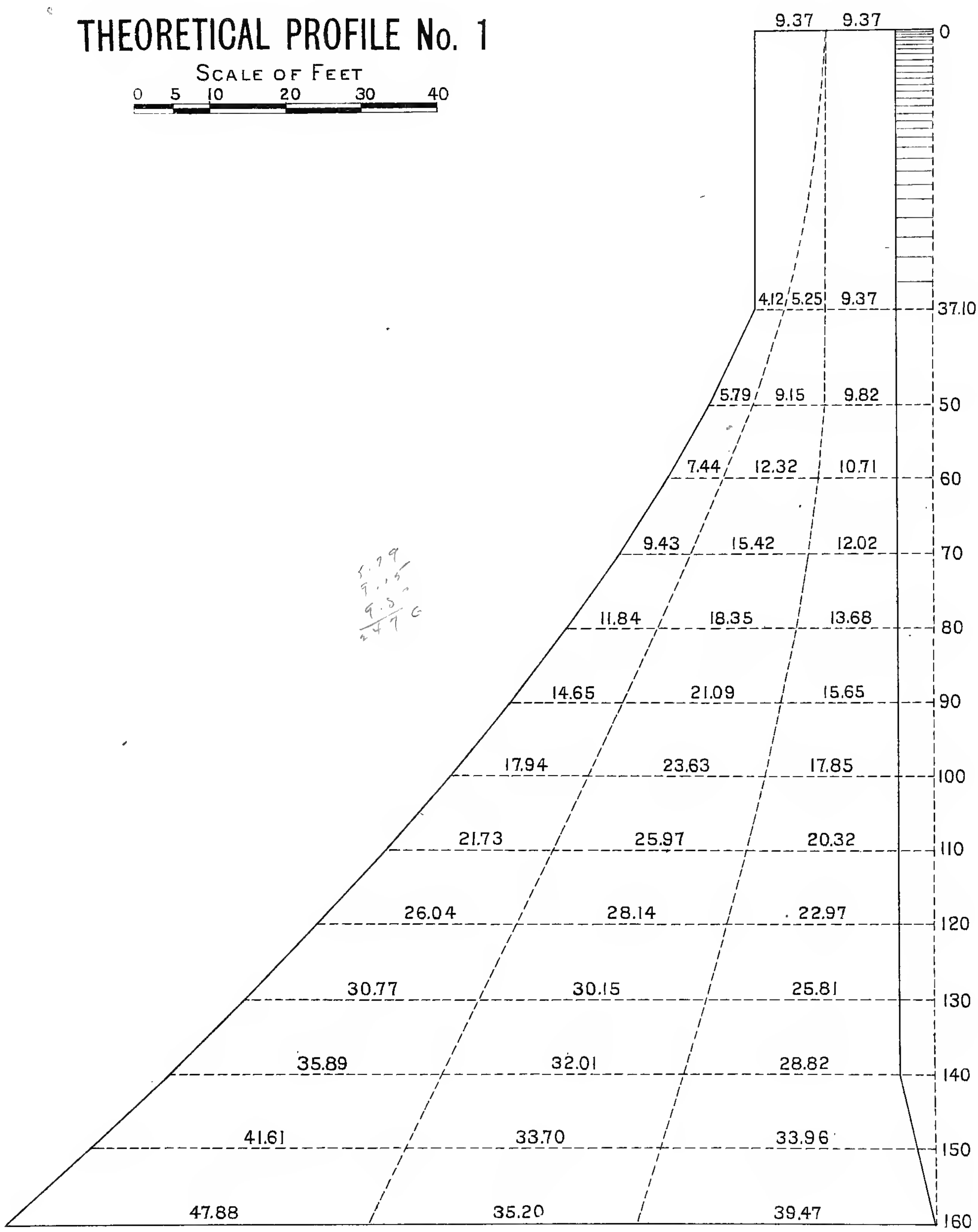

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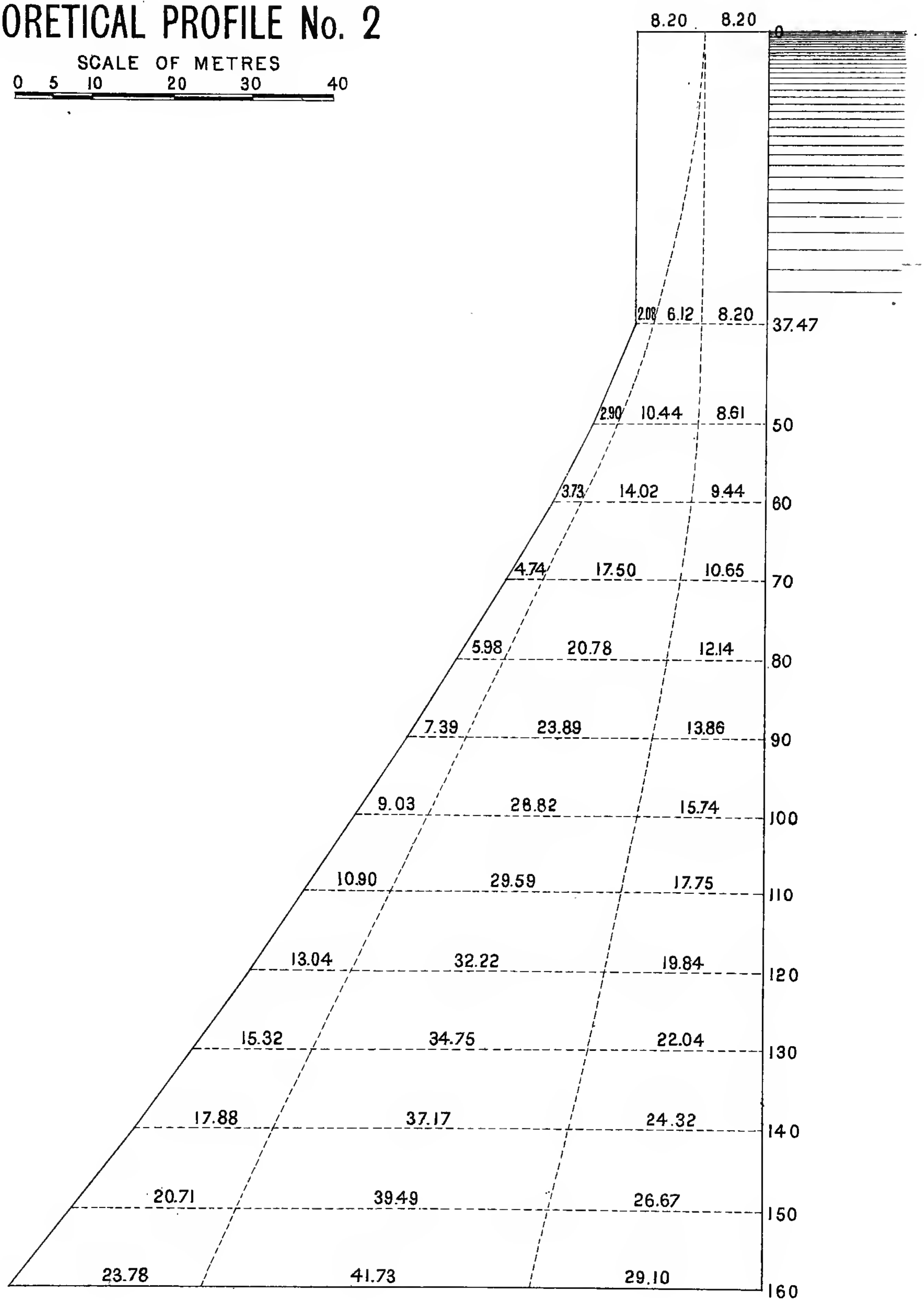

PLATE X.

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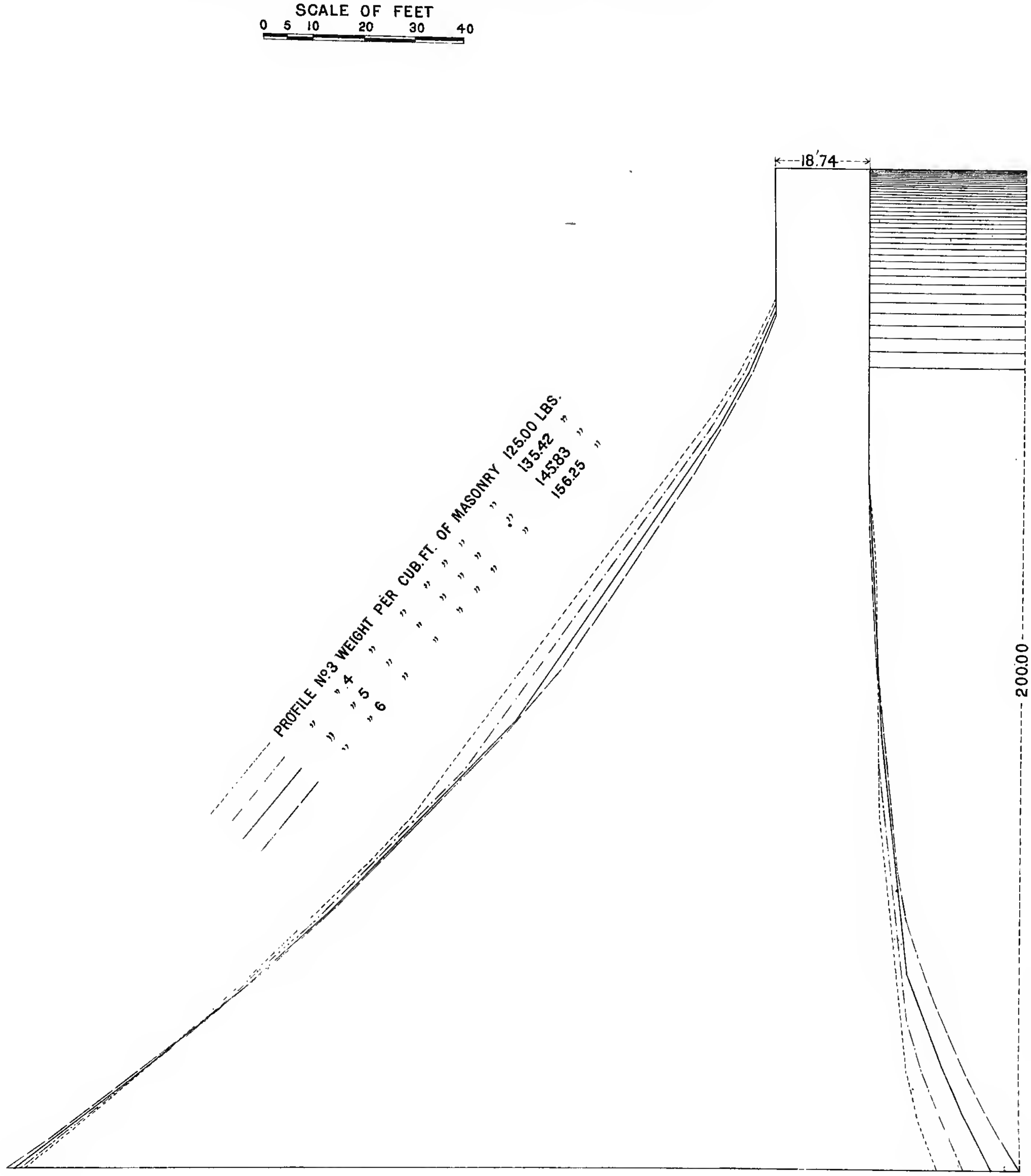
COMPARISON OF THEORETICAL PROFILES



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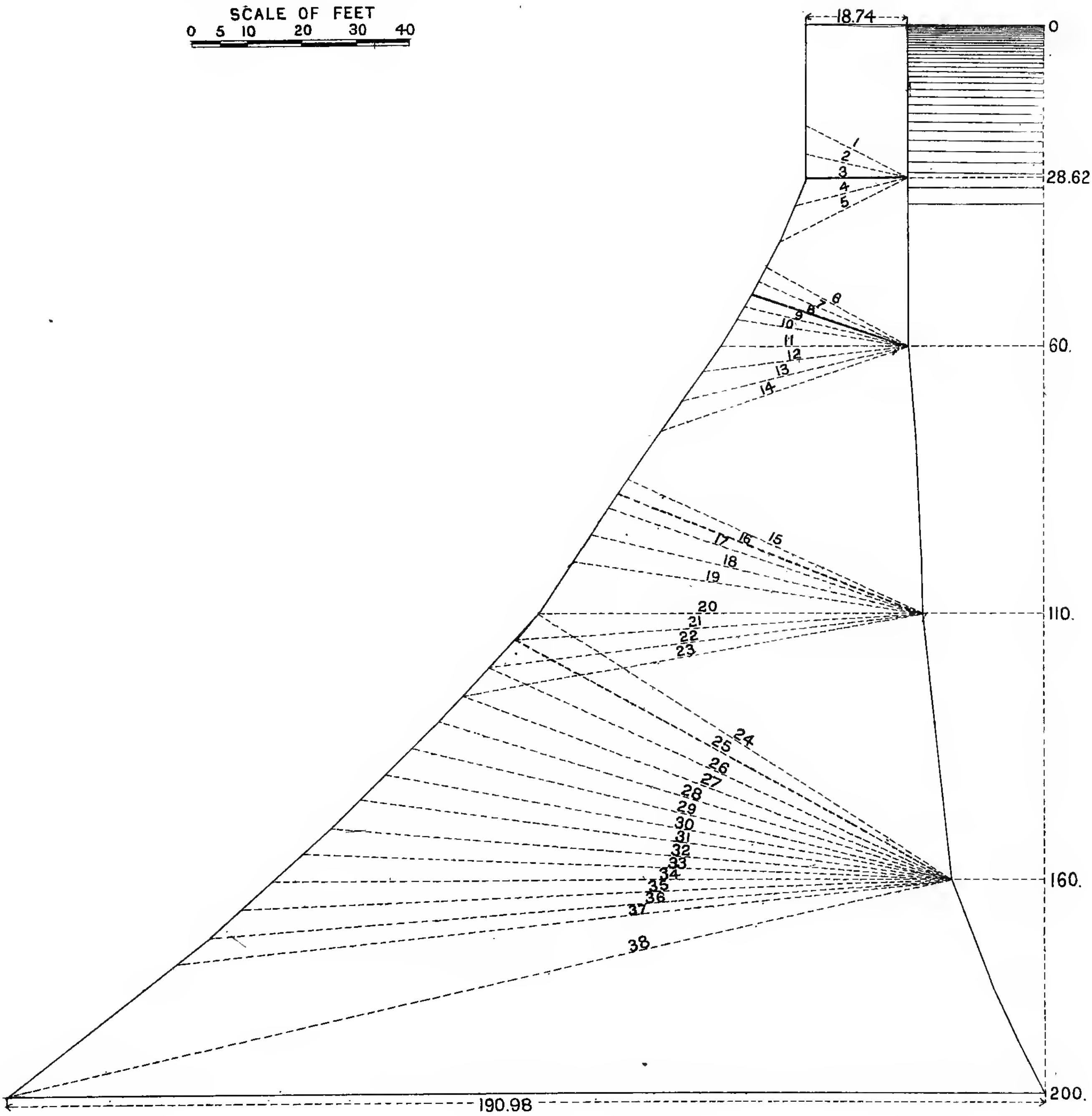

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M. BOUVIER'S FORMULÆ

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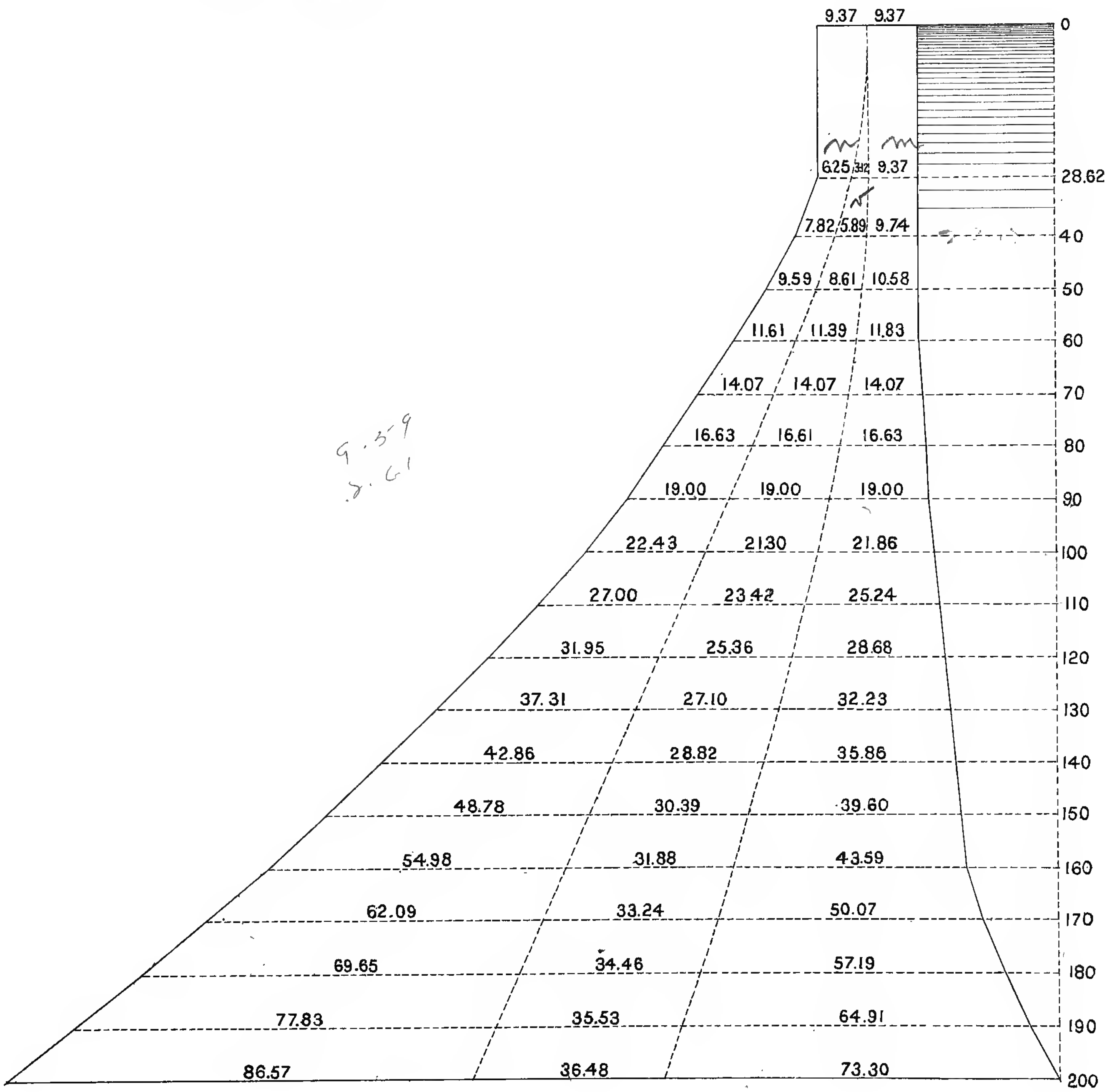

9.59
8.61

19 6.3
65

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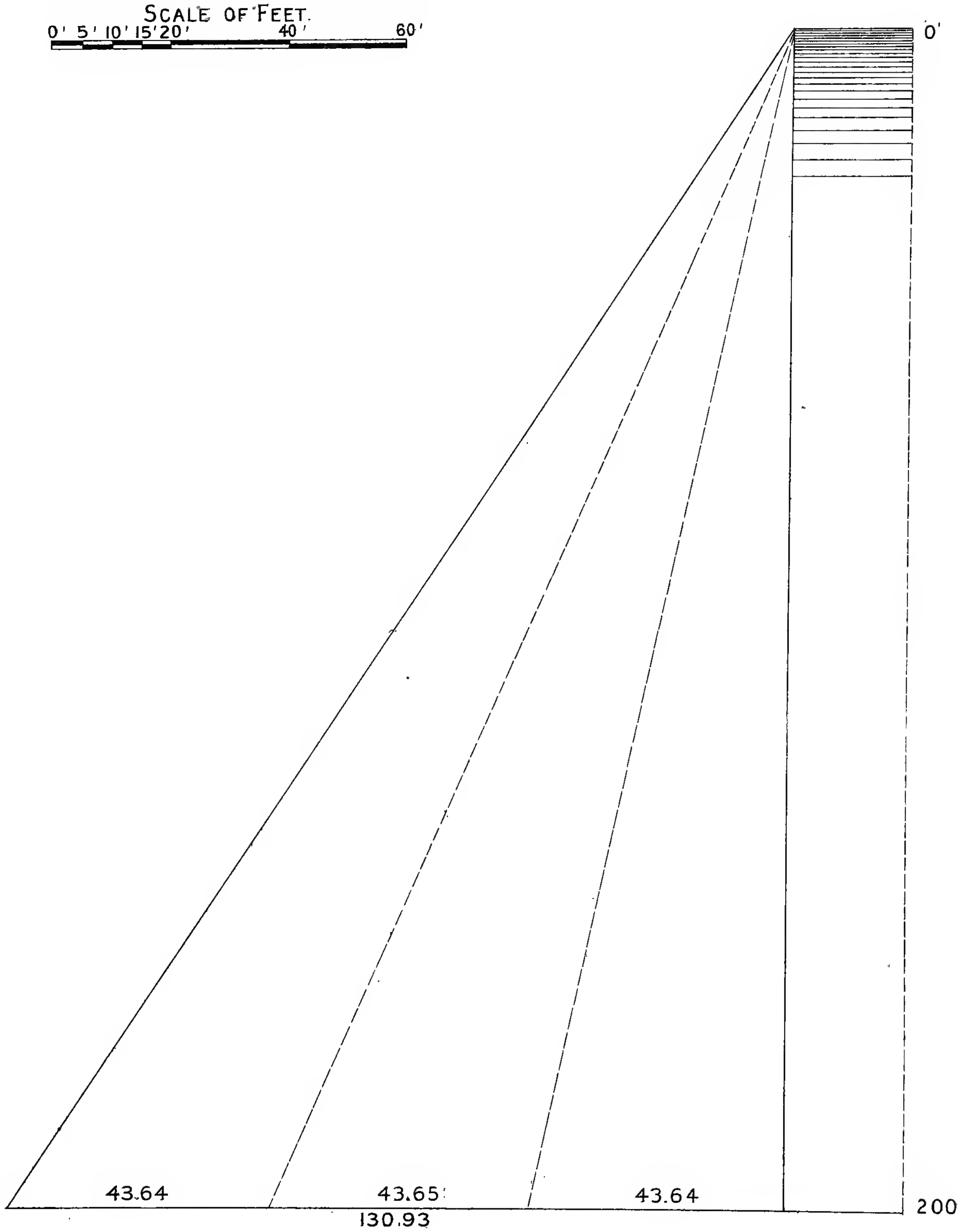

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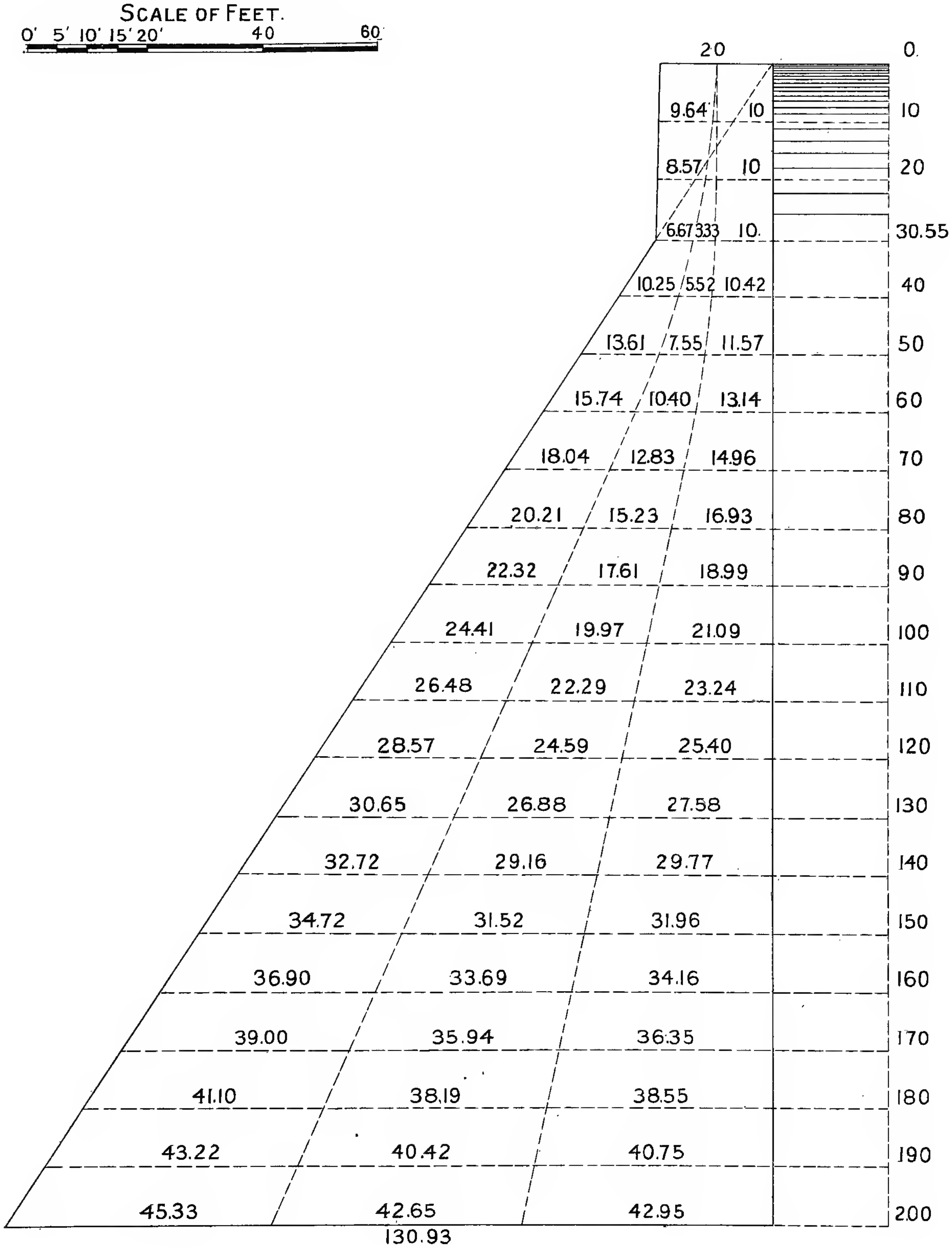

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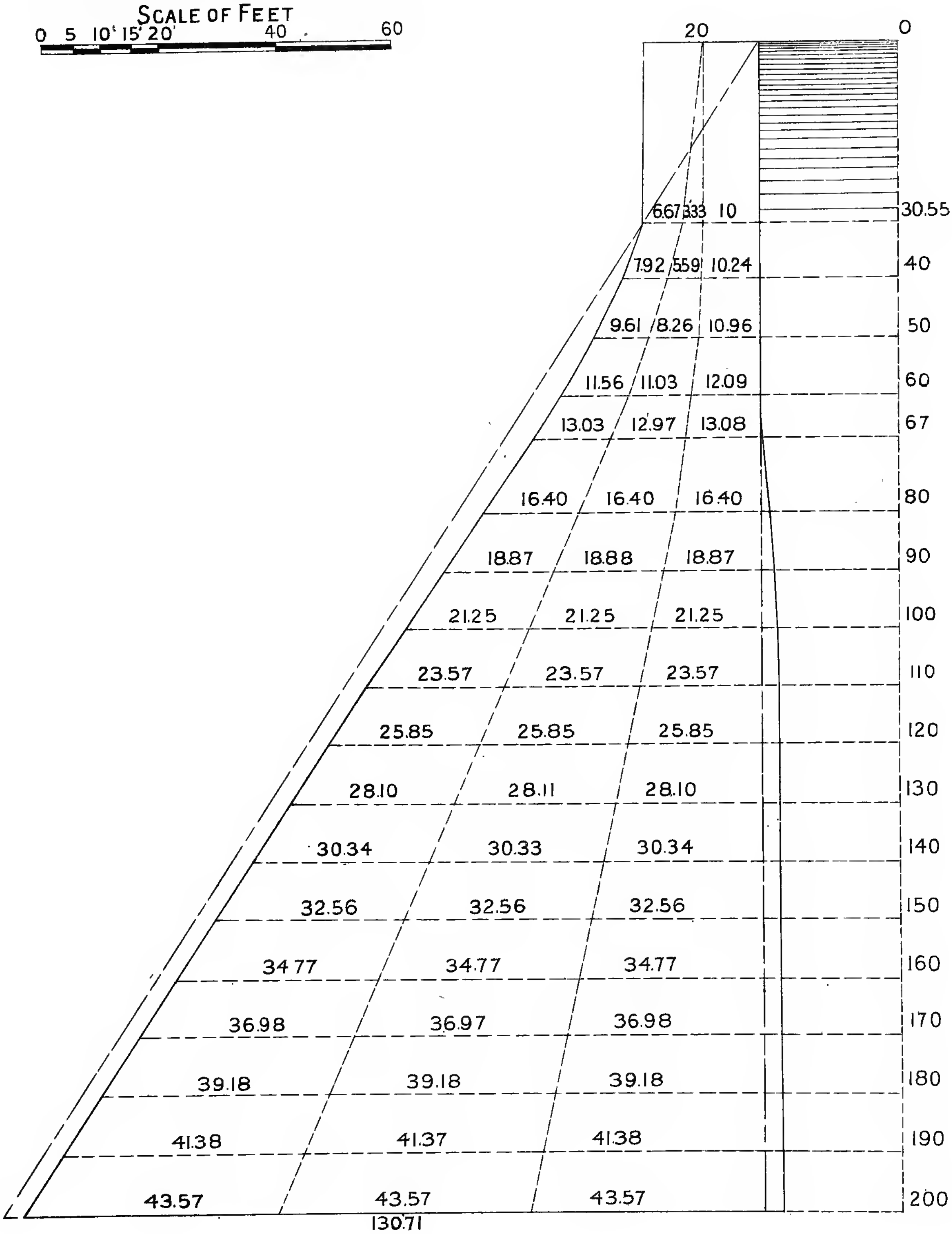

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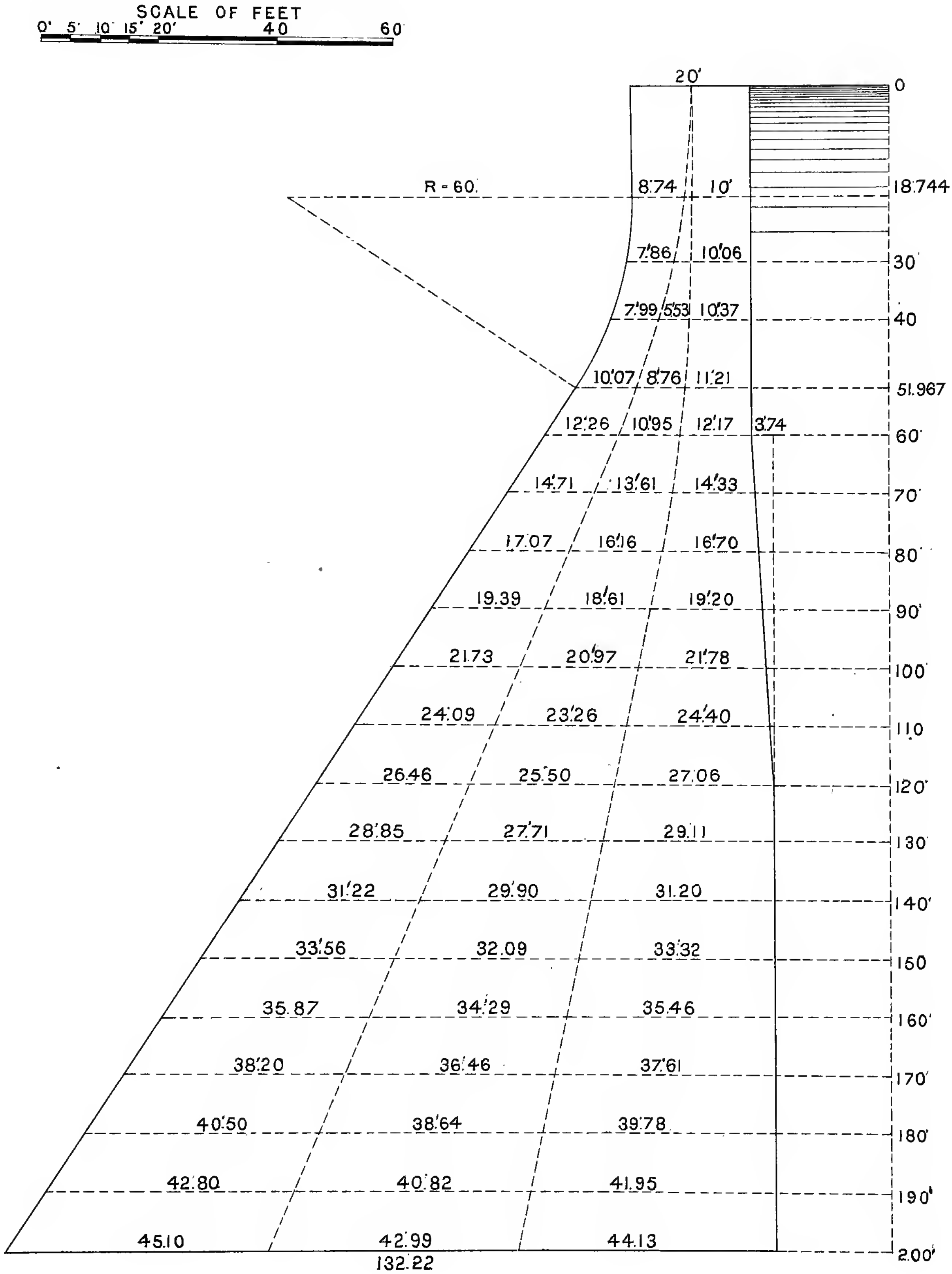

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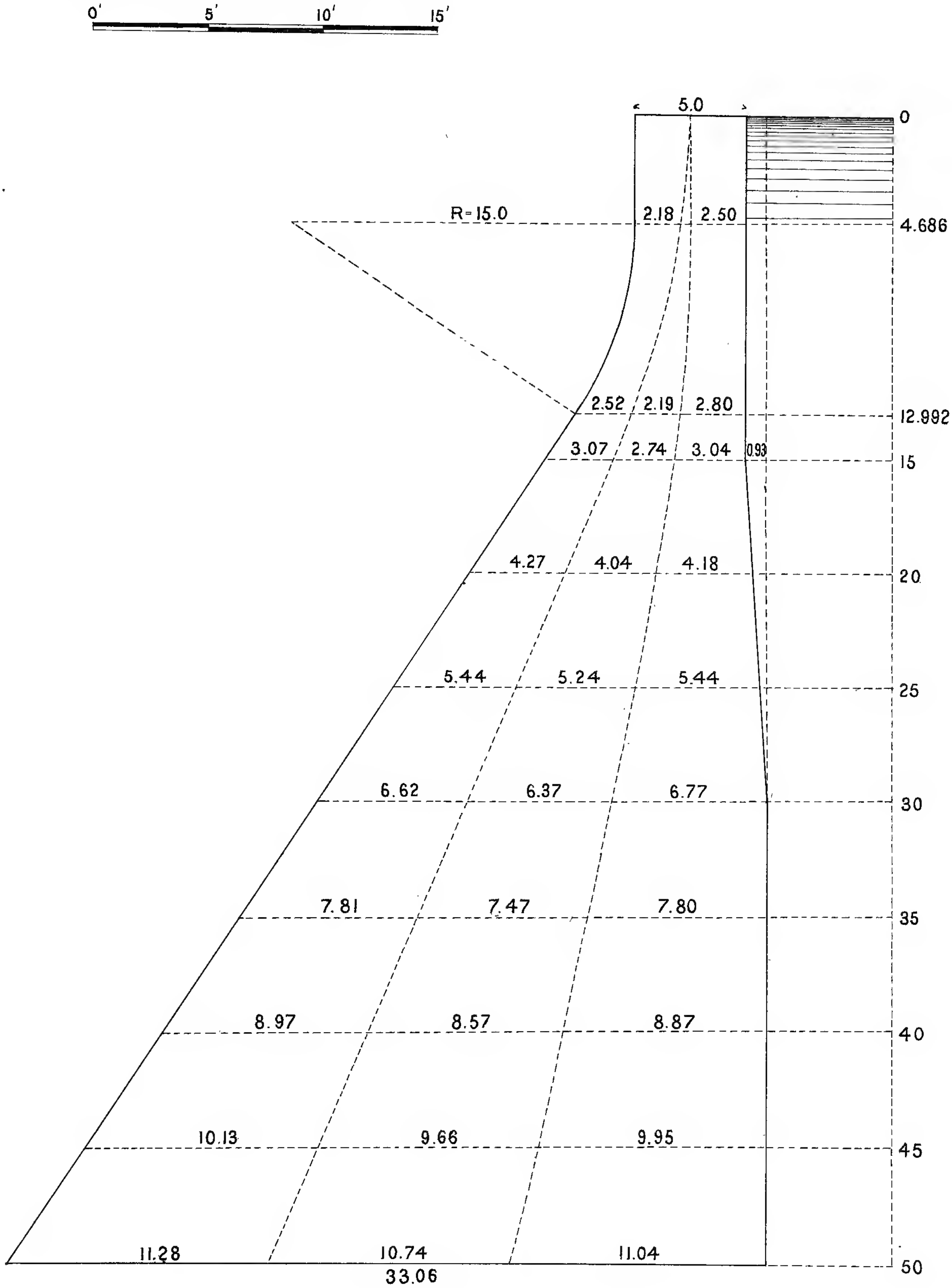

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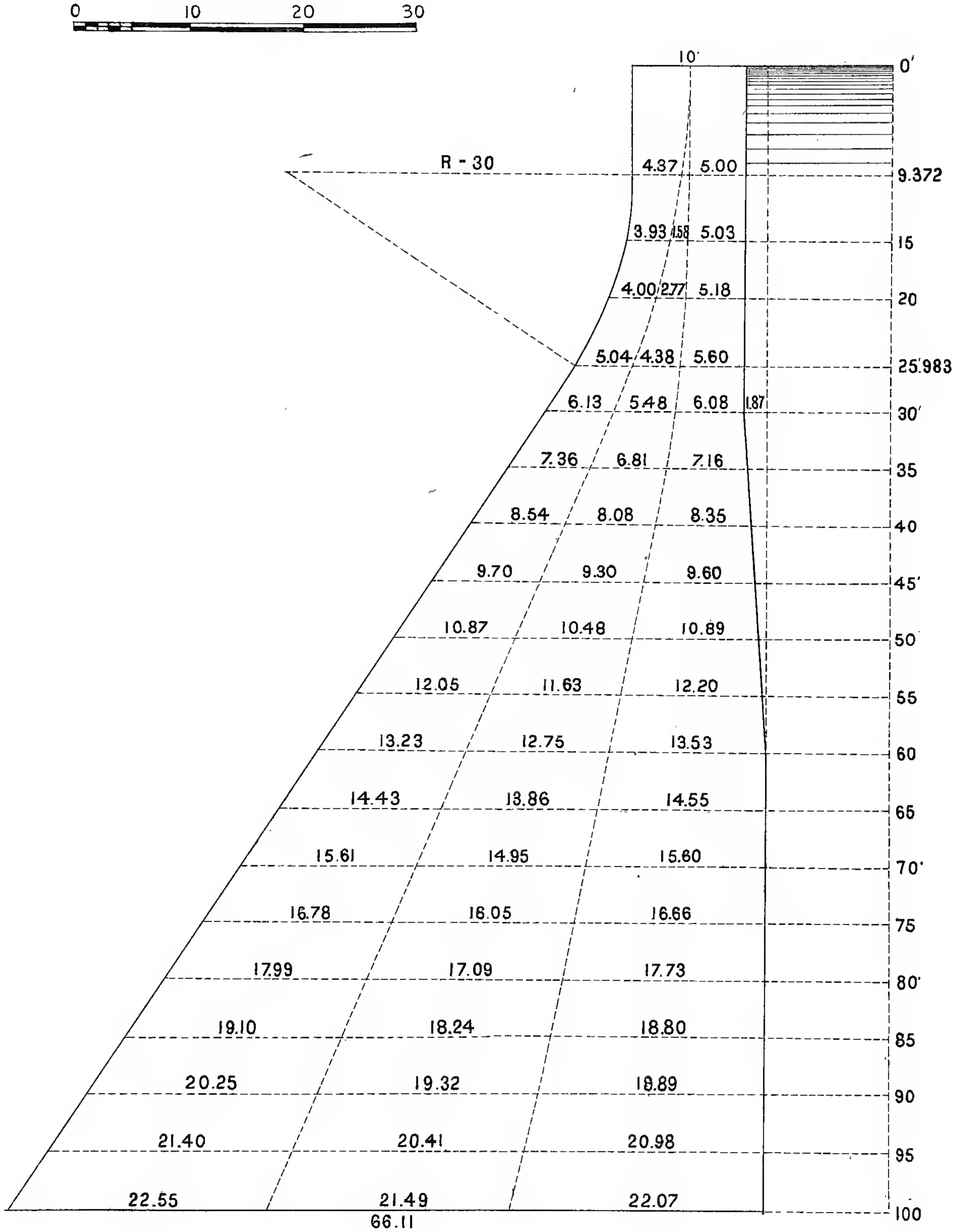

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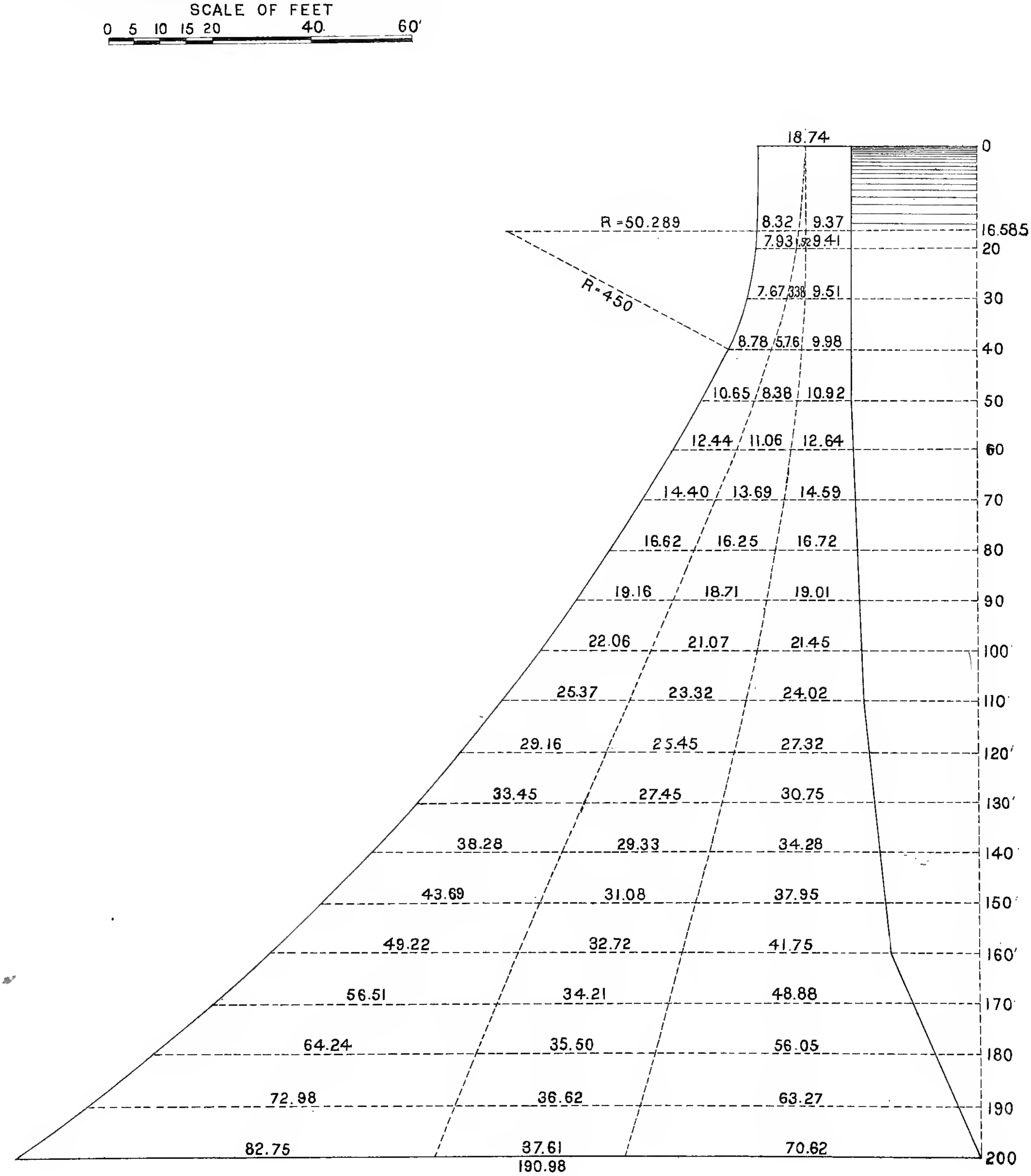

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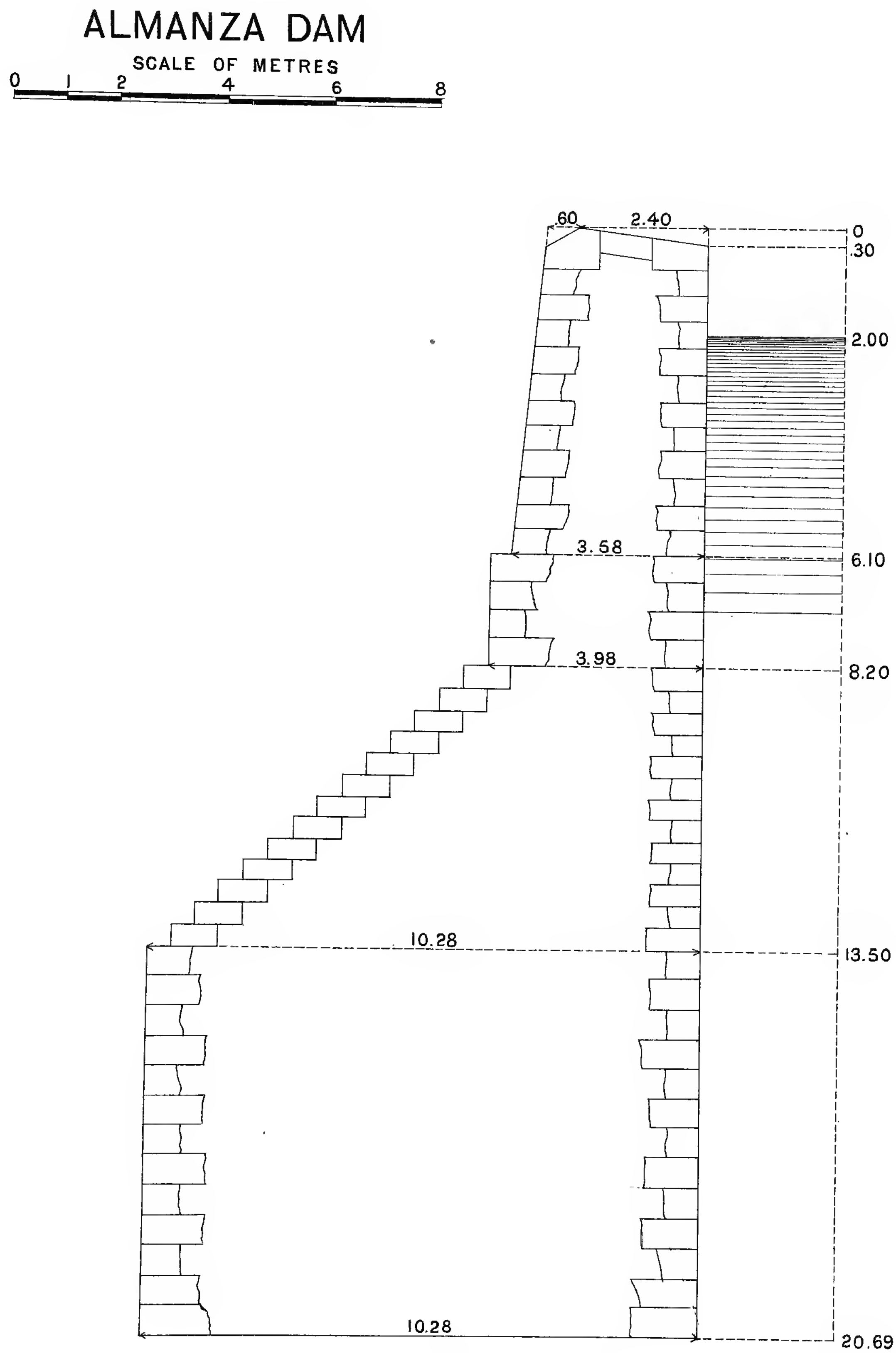
ALMANZA DAM
SCALE OF METRES
0
1
2
4
6
8
.60
2.40
0
.30
2.00
3.58
6.10
3.98
8.20
10.28
13.50
10.28
20.69

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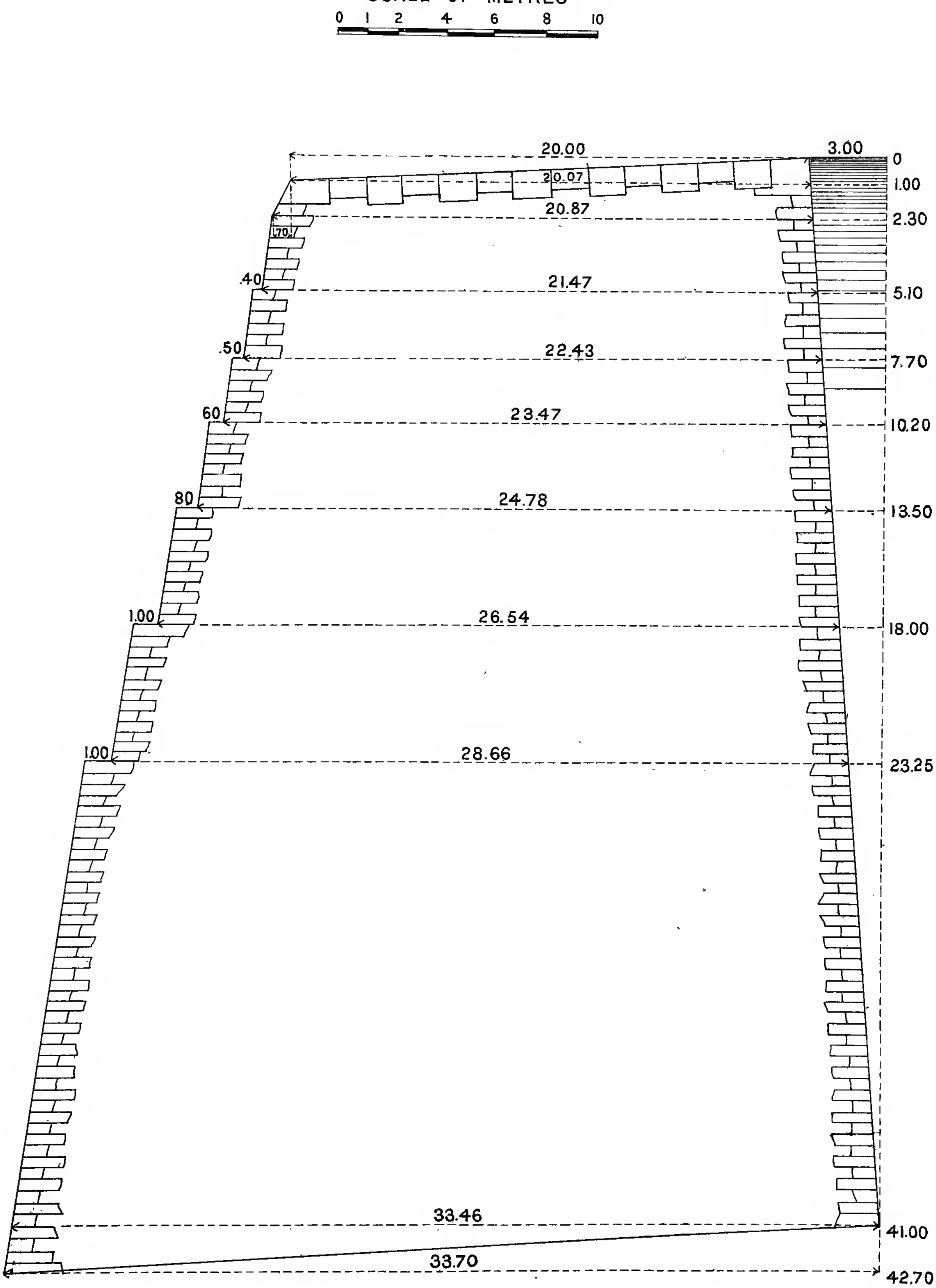
ALICANTE DAM



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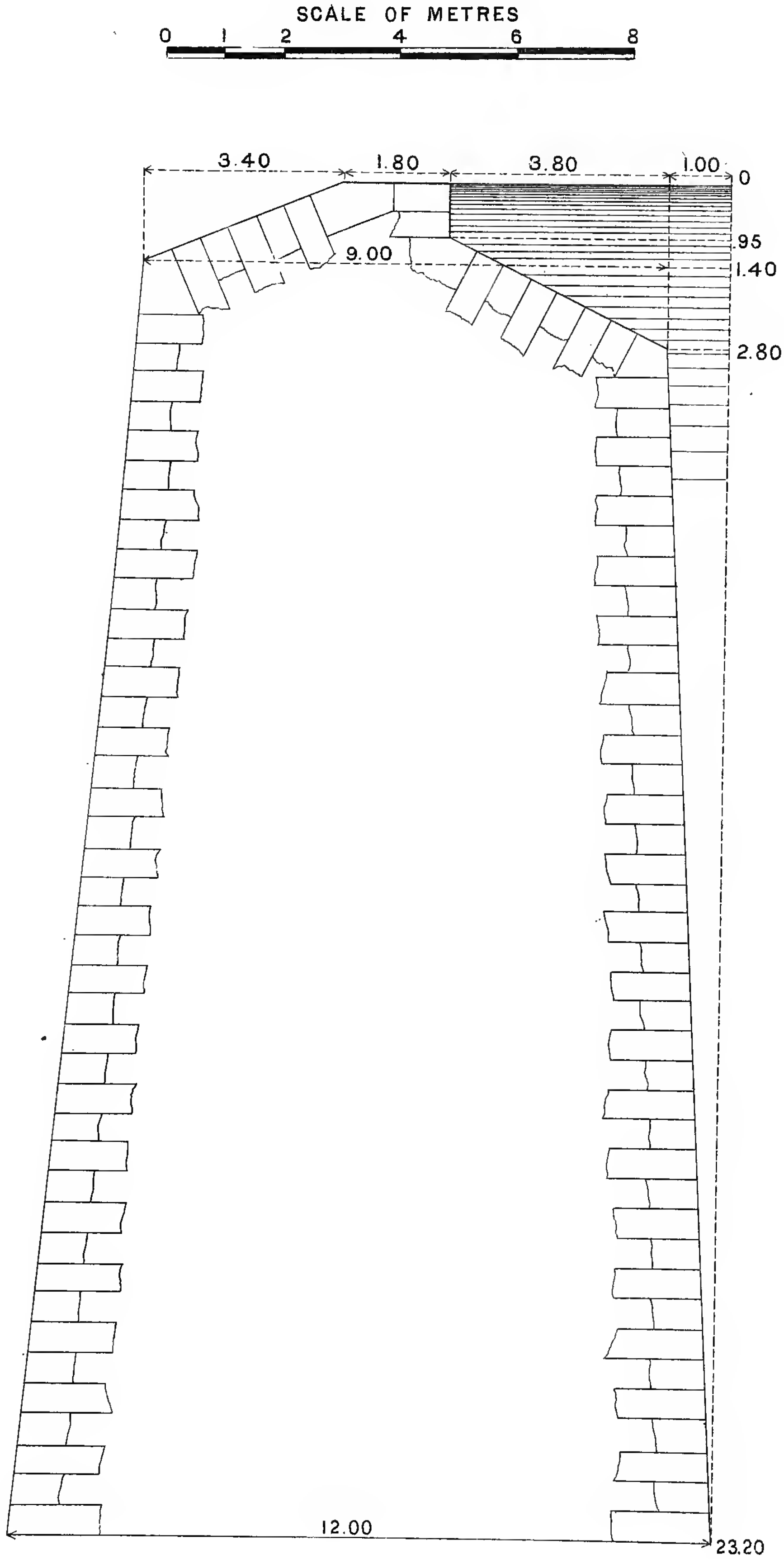
ELCHE DAM



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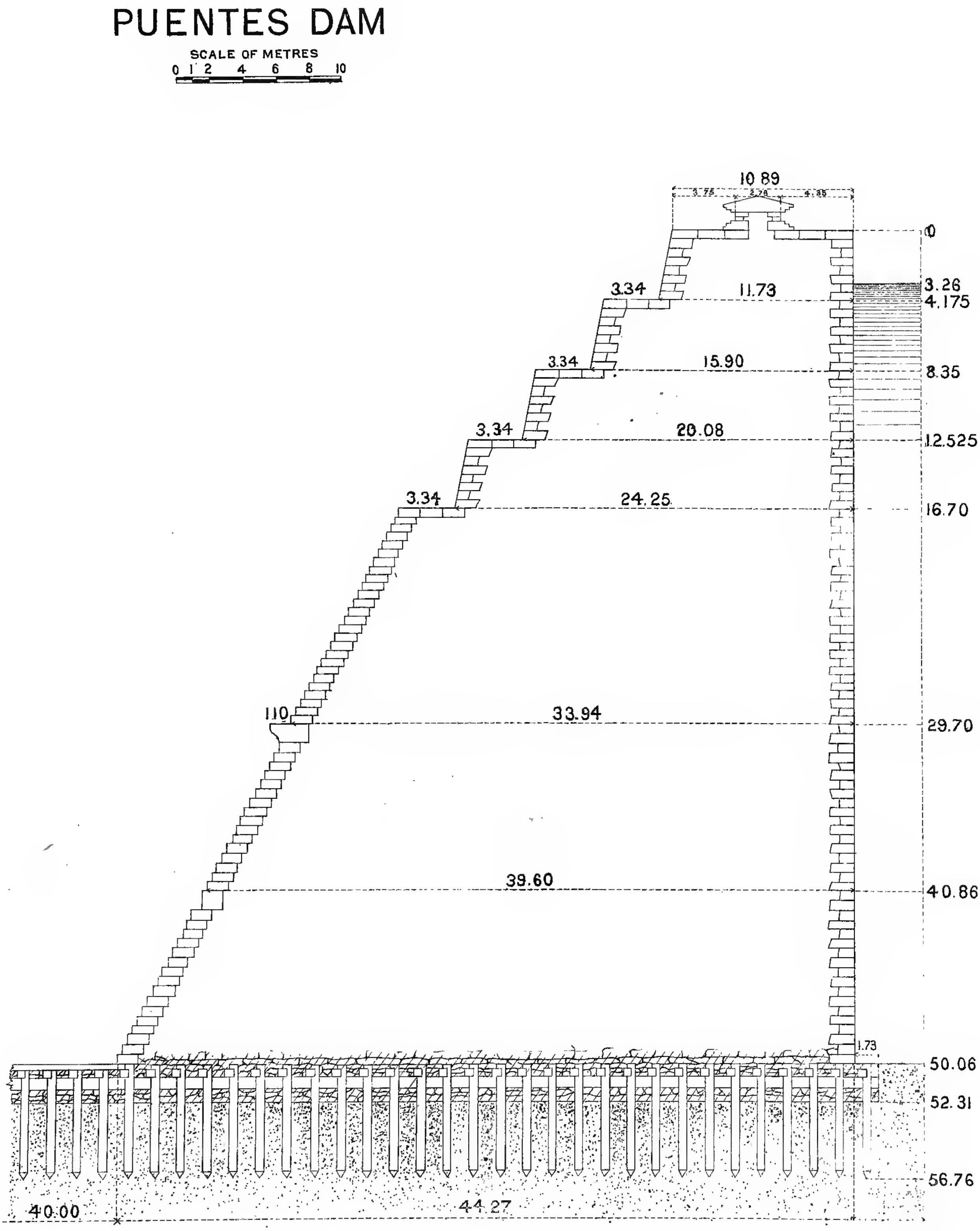

PLATE XXV.

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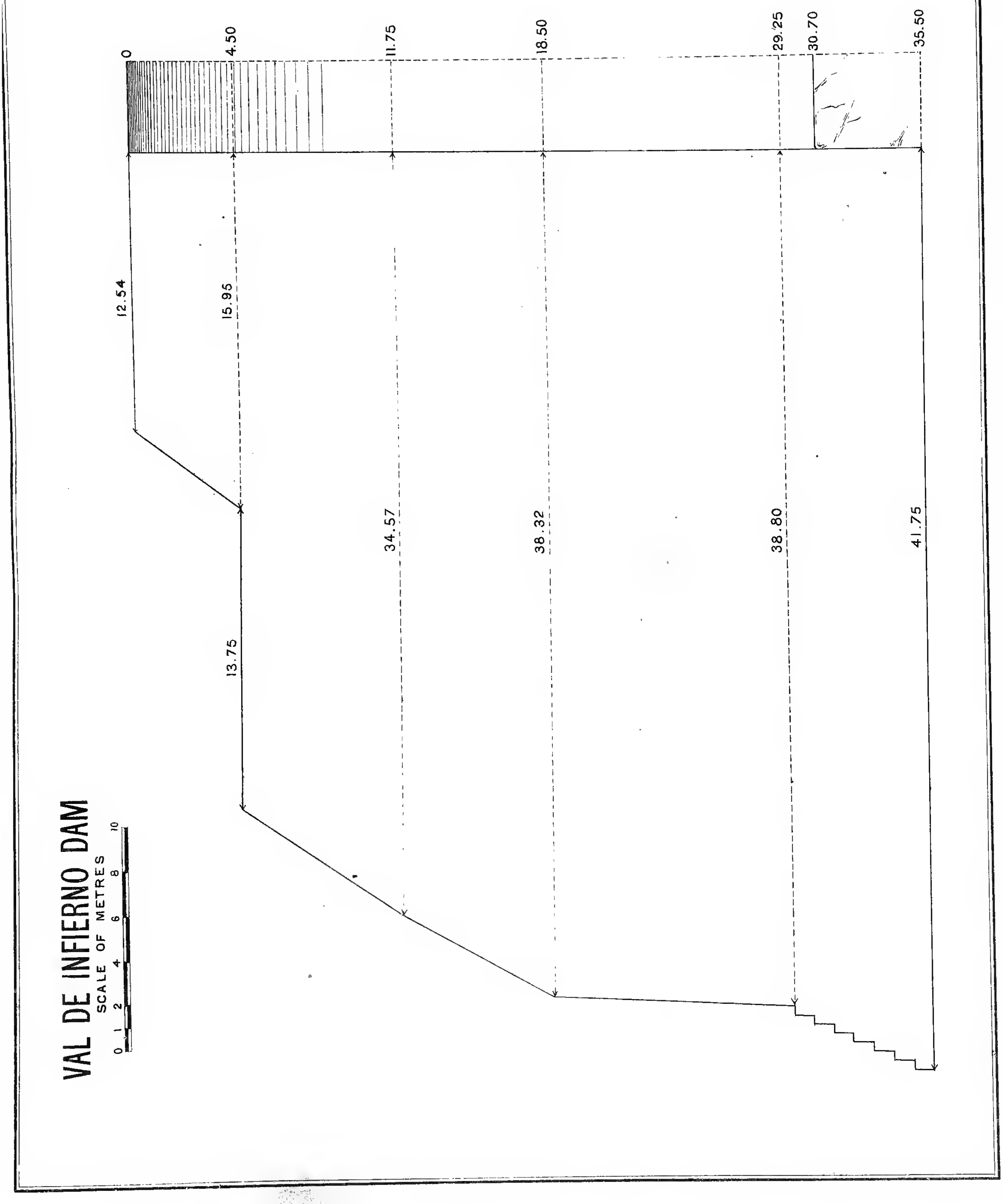

VAL DE INFIERNO DAM

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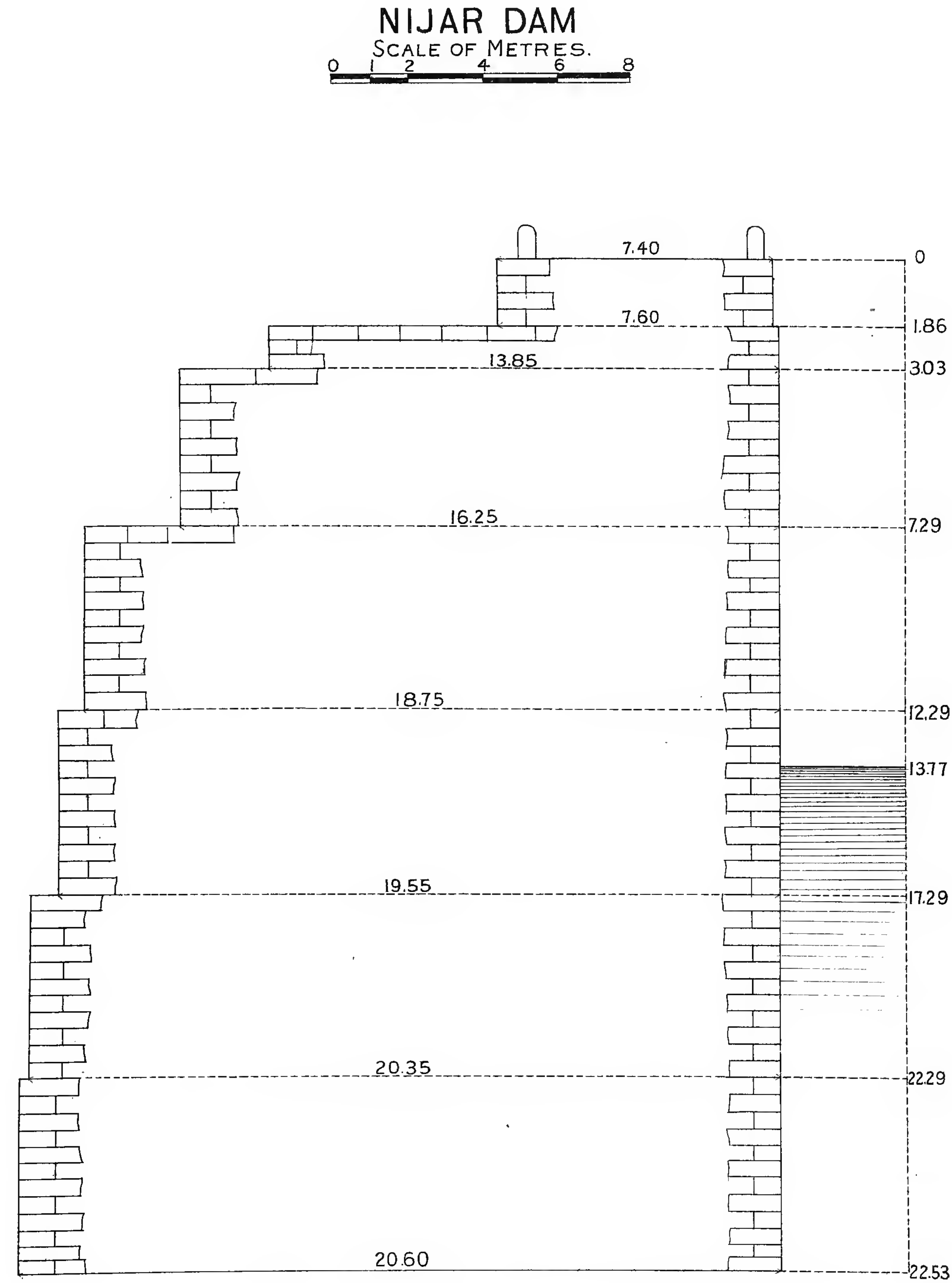
NIJAR DAM
SCALE OF METRES.
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7.40
0
7.60
1.86
13.85
3.03
16.25
7.29
18.75
12.29
13.77
19.55
17.29
20.35
22.29
20.60
22.53

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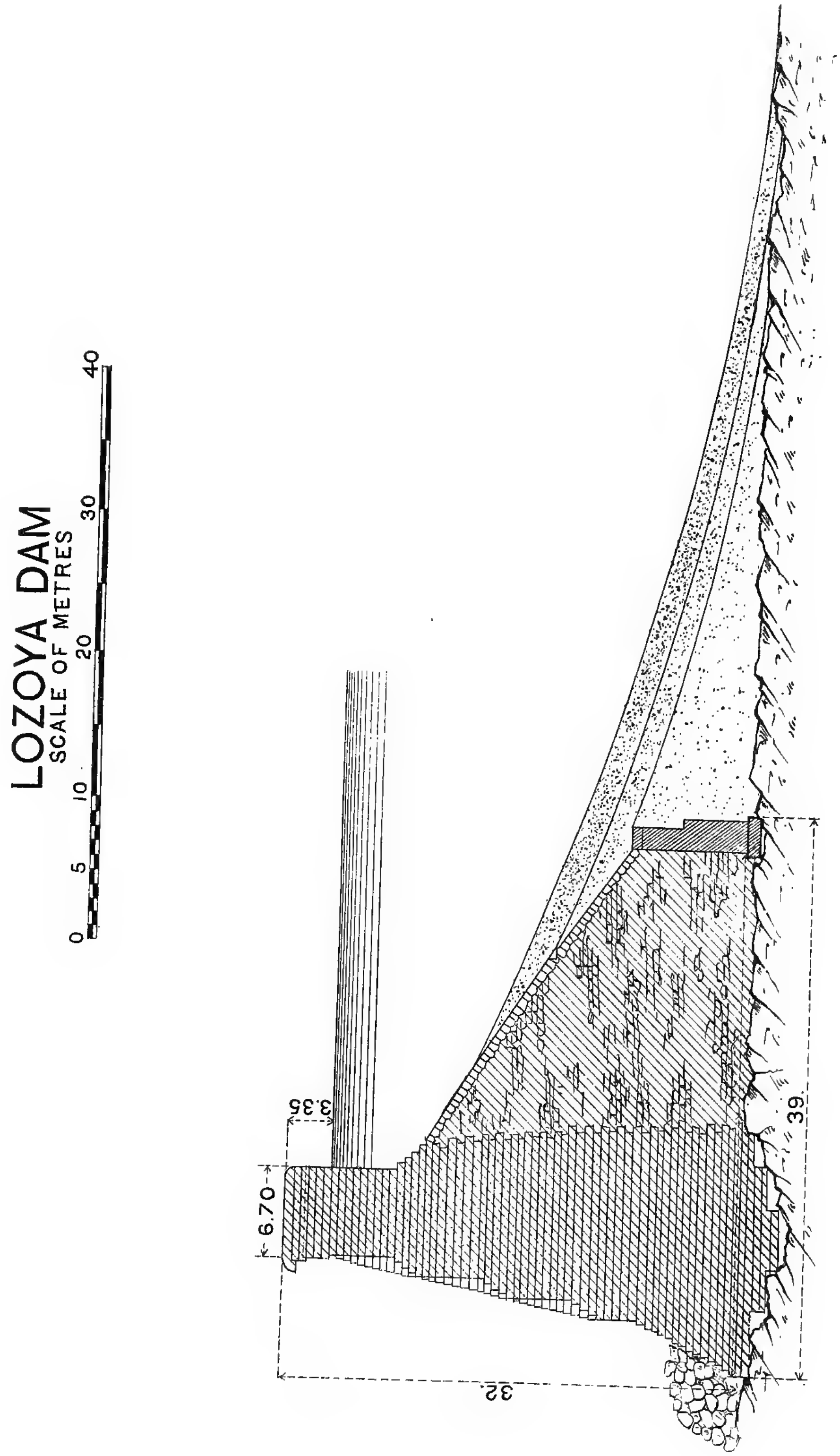

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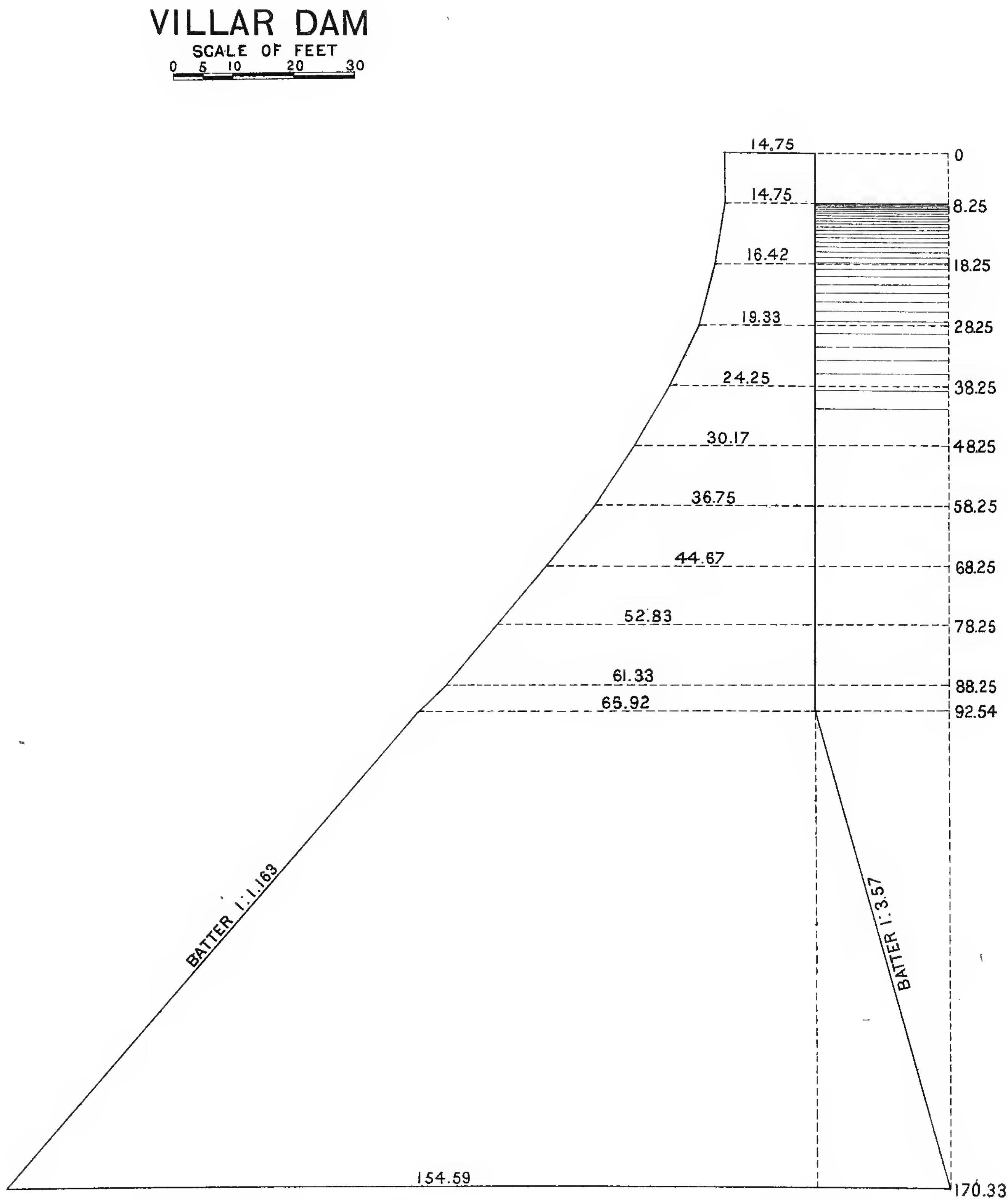
VILLAR DAM
SCALE OF FEET
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14.75
16.42
19.33
24.25
30.17
36.75
44.67
52.83
61.33
65.92
154.59
0
8.25
18.25
28.25
38.25
48.25
58.25
68.25
78.25
88.25
92.54
170.33
BATTER 1:1.163
BATTER 1:3.57

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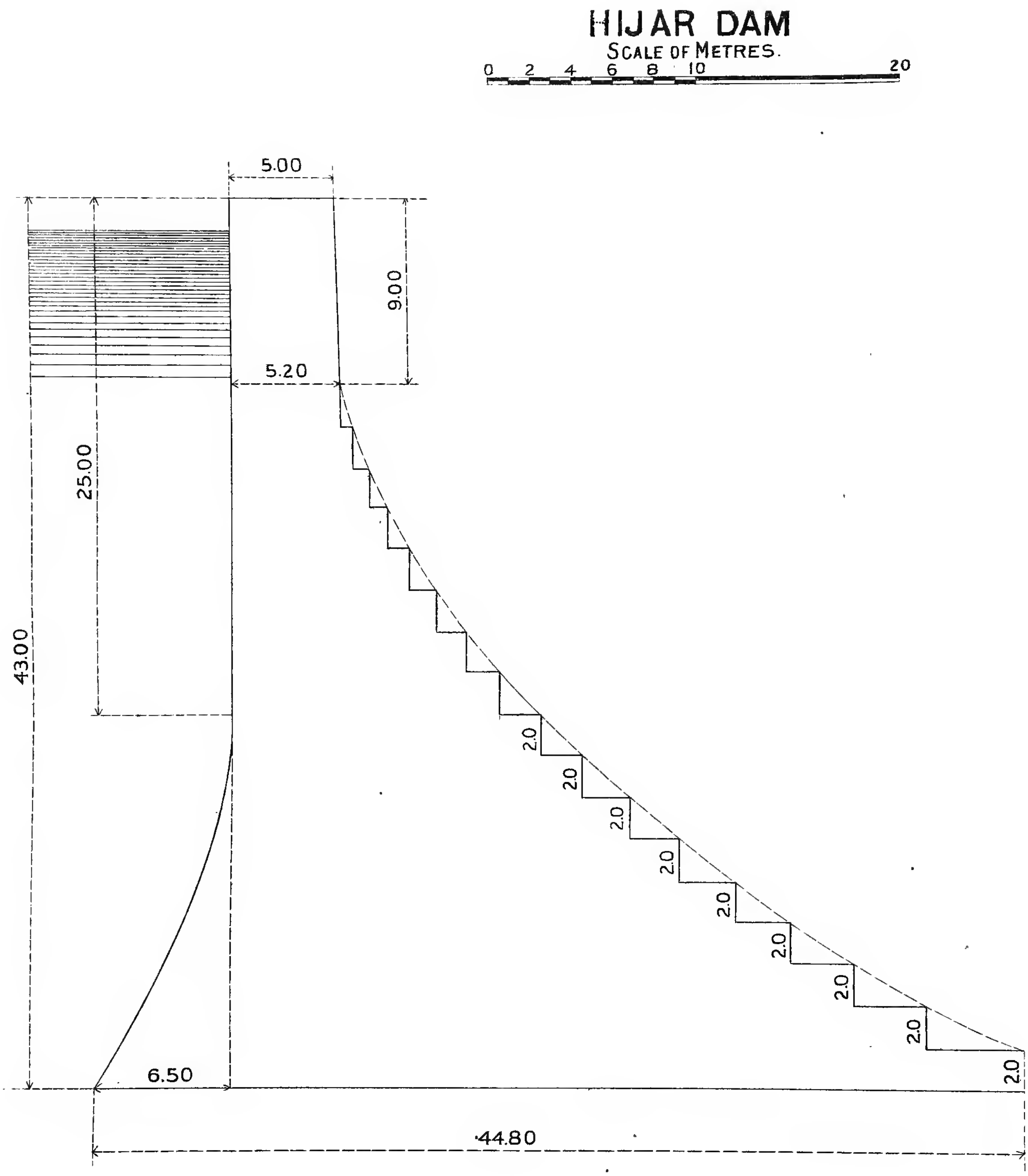
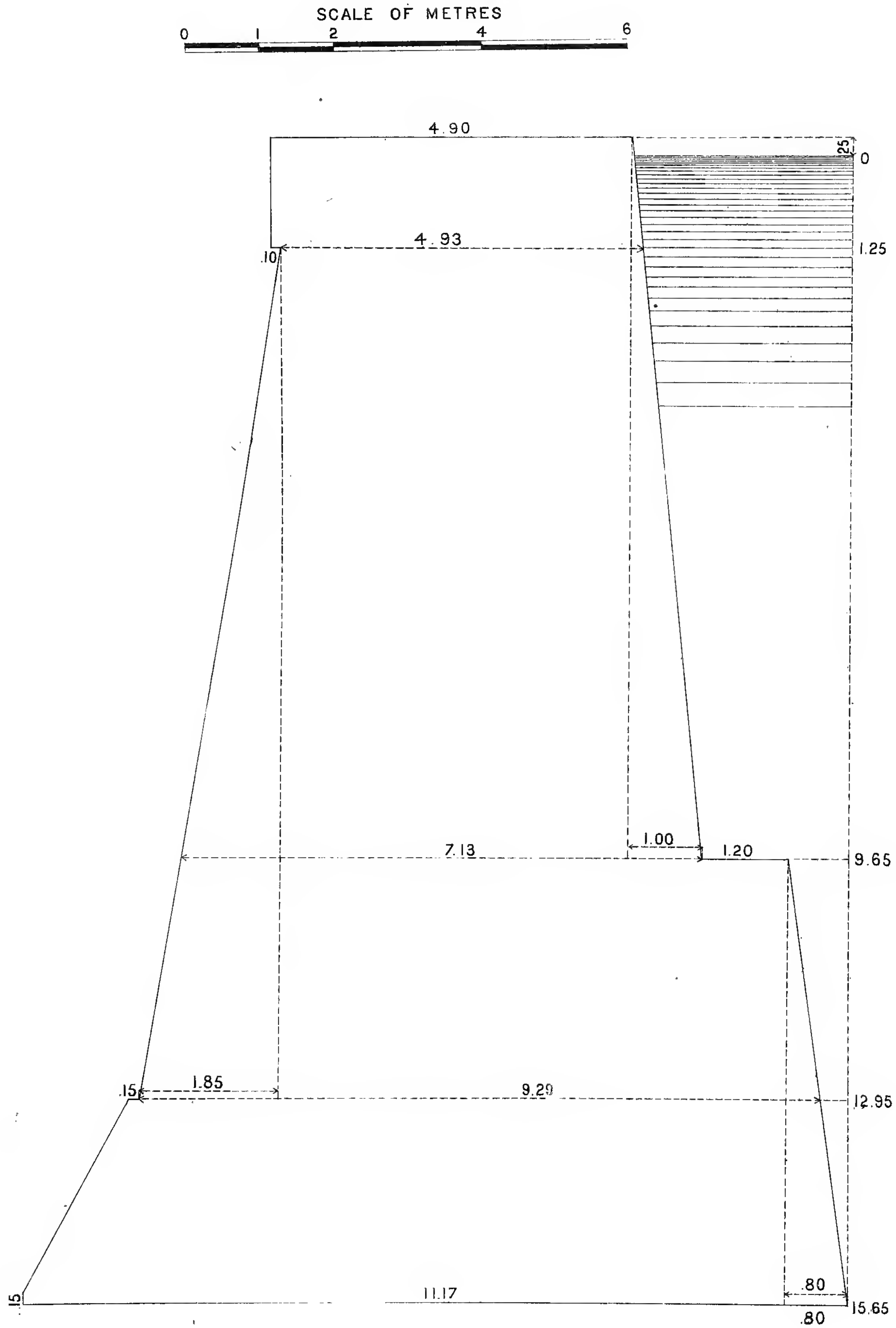

HIJAR DAM
SCALE OF METRES.
0 2 4 6 8 10 20
5.00
9.00
5.20
25.00
43.00
2.0
2.0
2.0
2.0
2.0
2.0
2.0
2.0
2.0
6.50
44.80

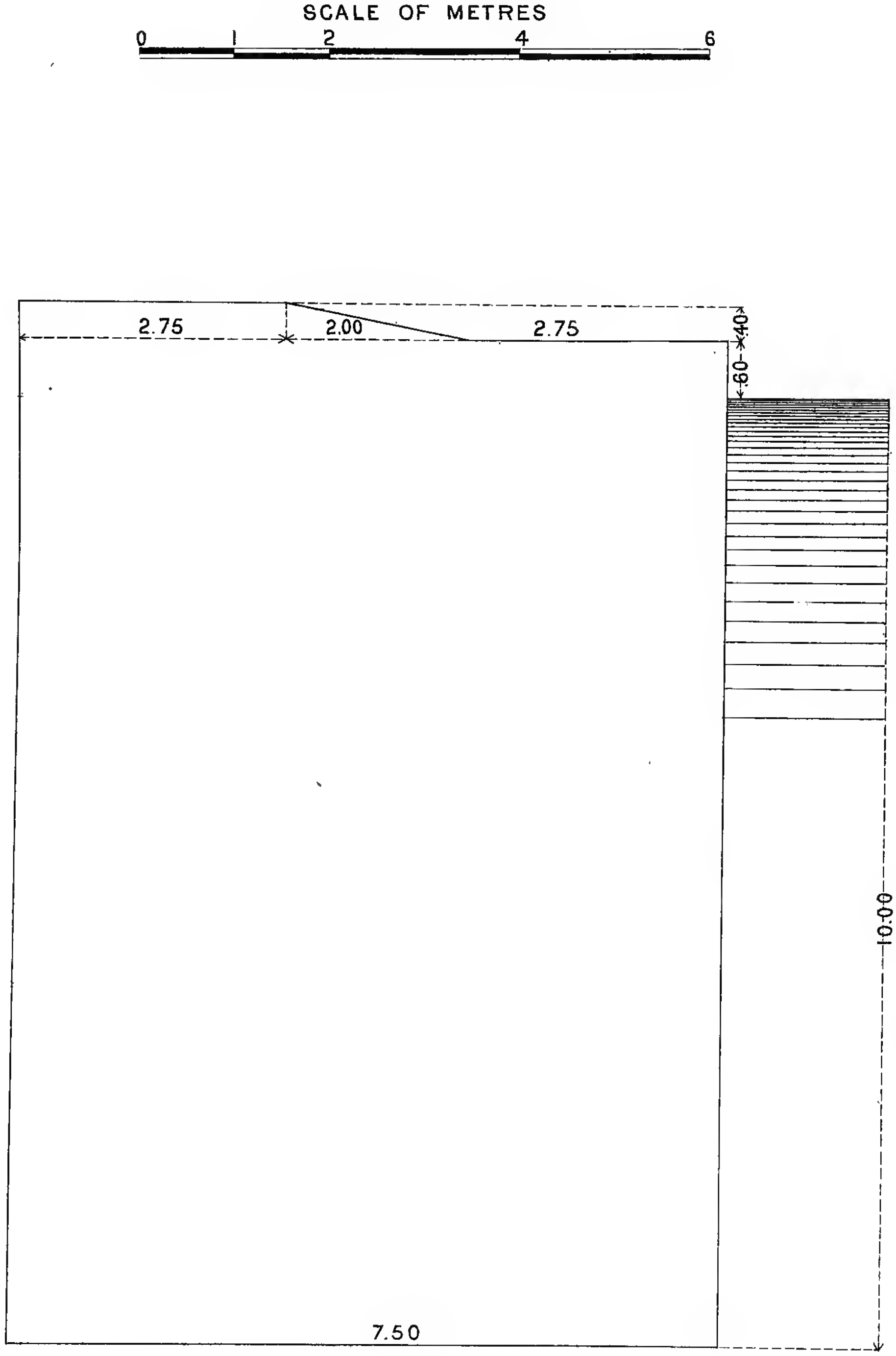
See Page 52

LAMPY DAM



SEE PAGE 52.

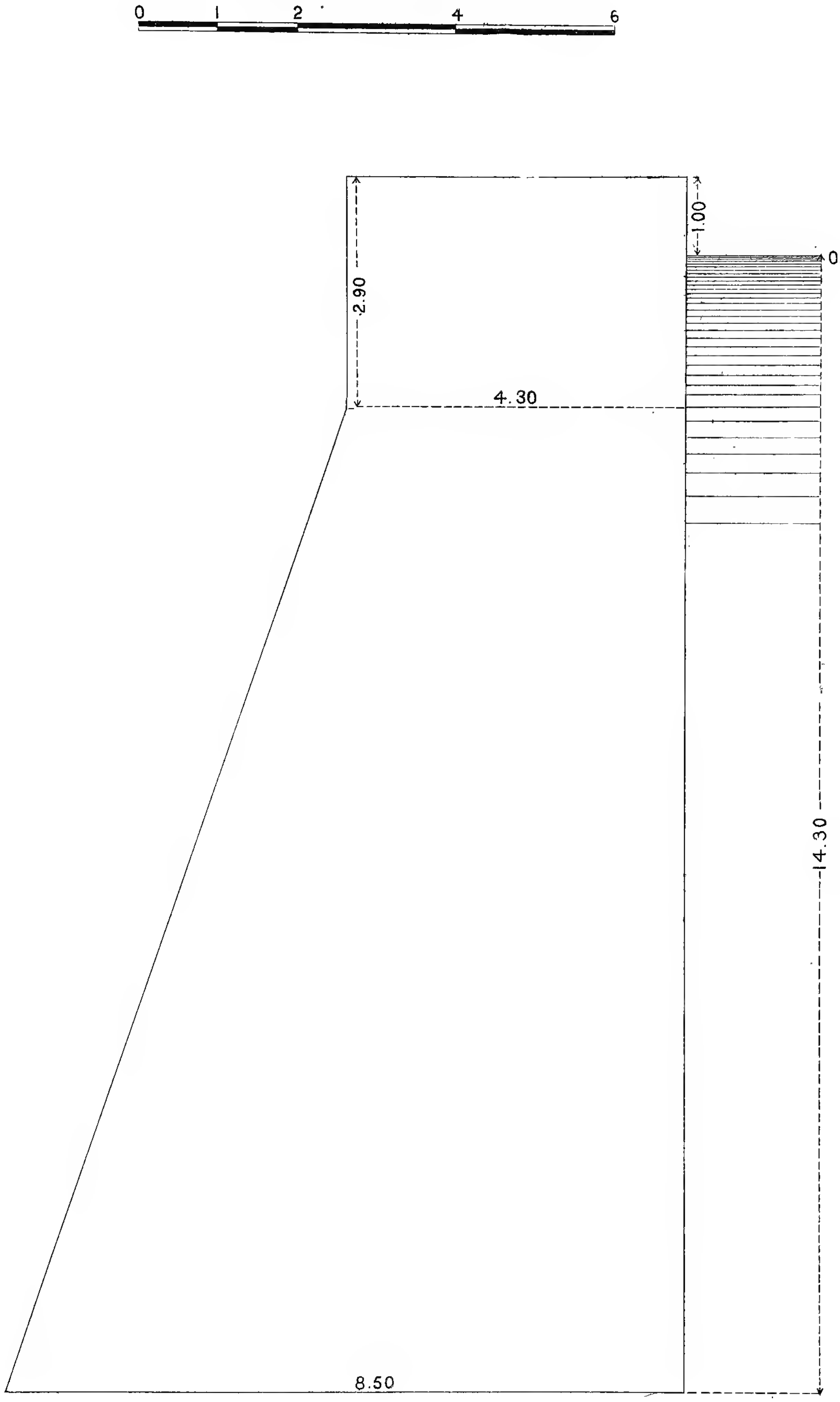
VIOREU DAM



See Page 52

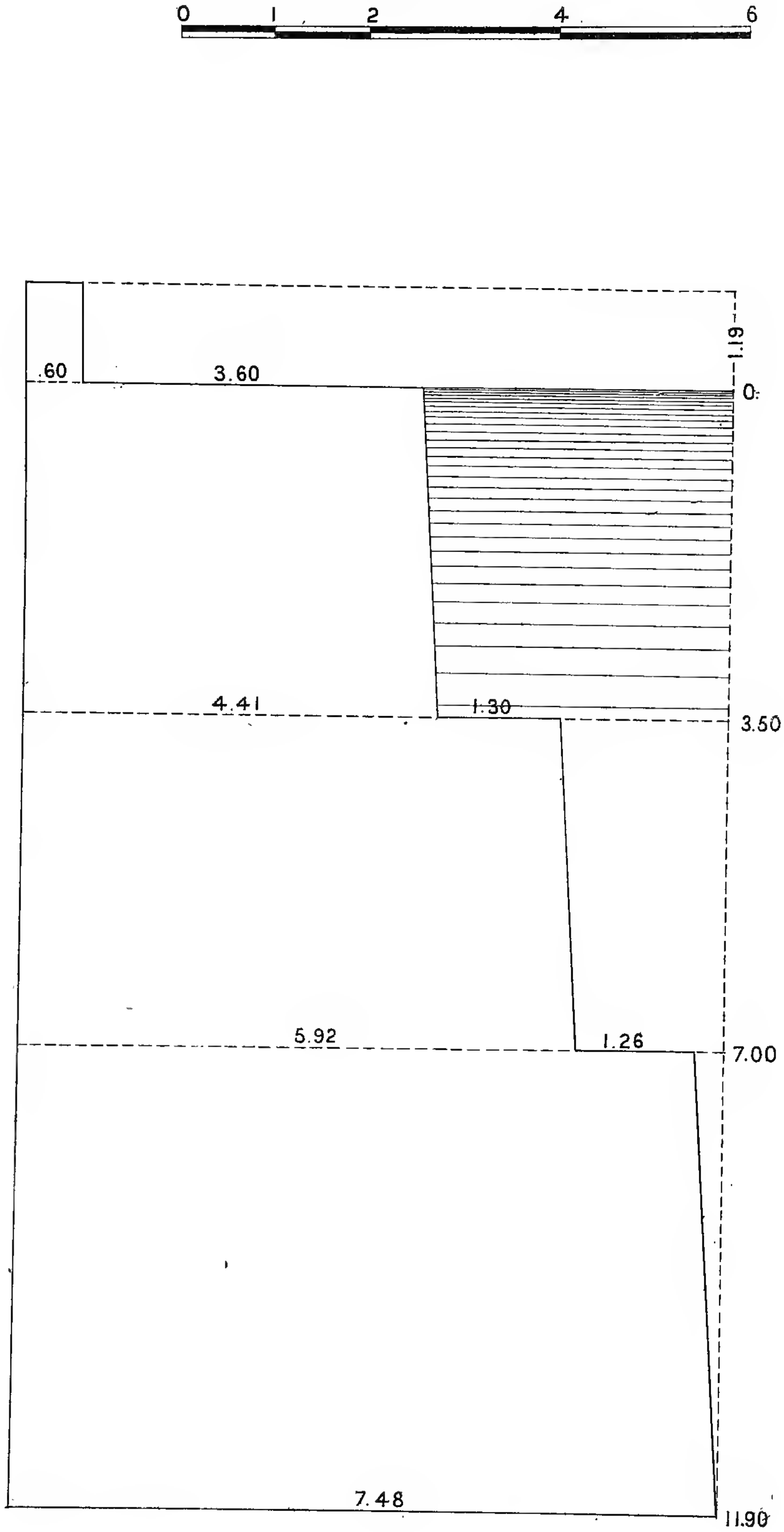
BOSMELEA DAM

SCALE OF METRES



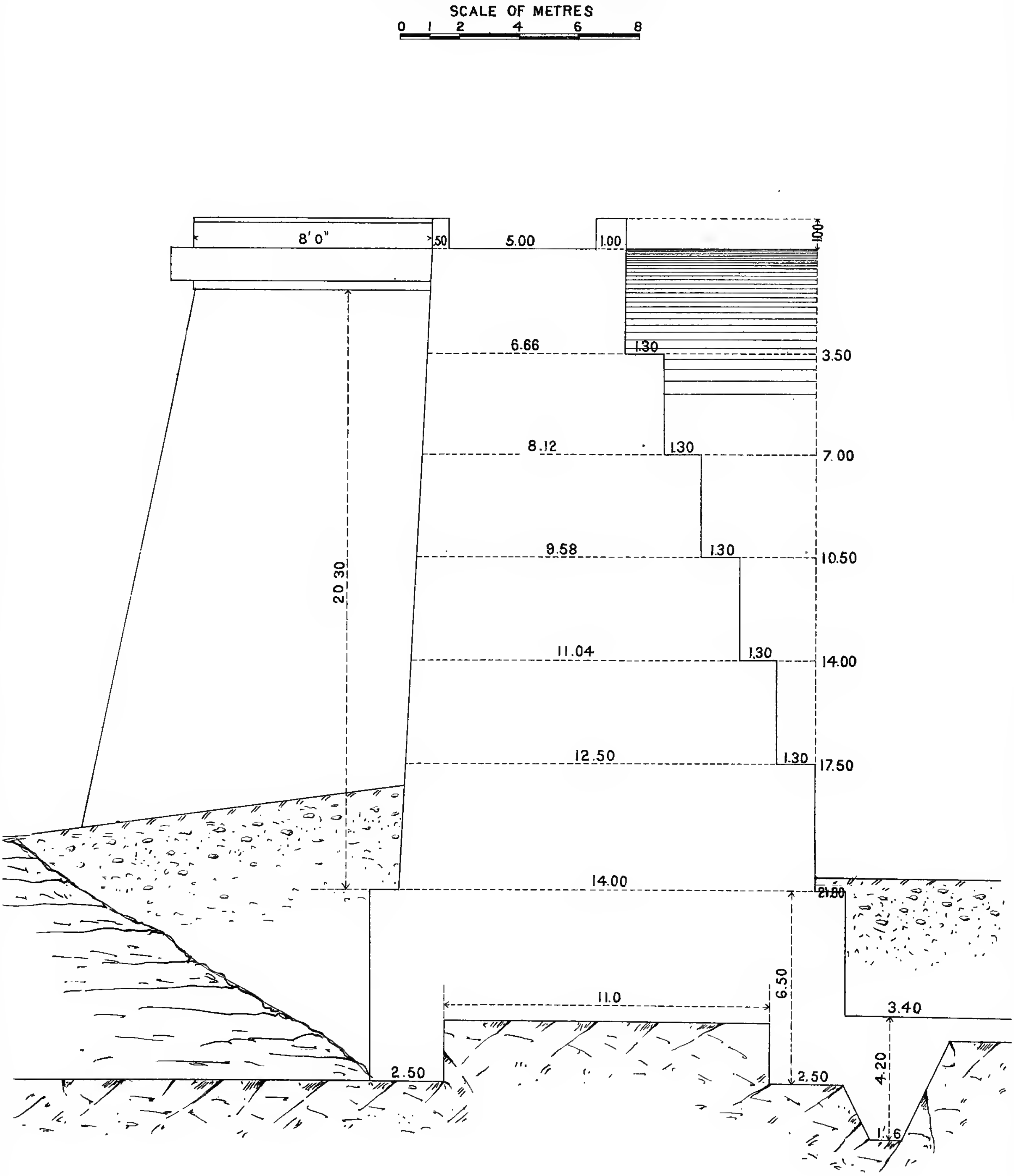
SEE PAGE 52

GLOMEL DAM



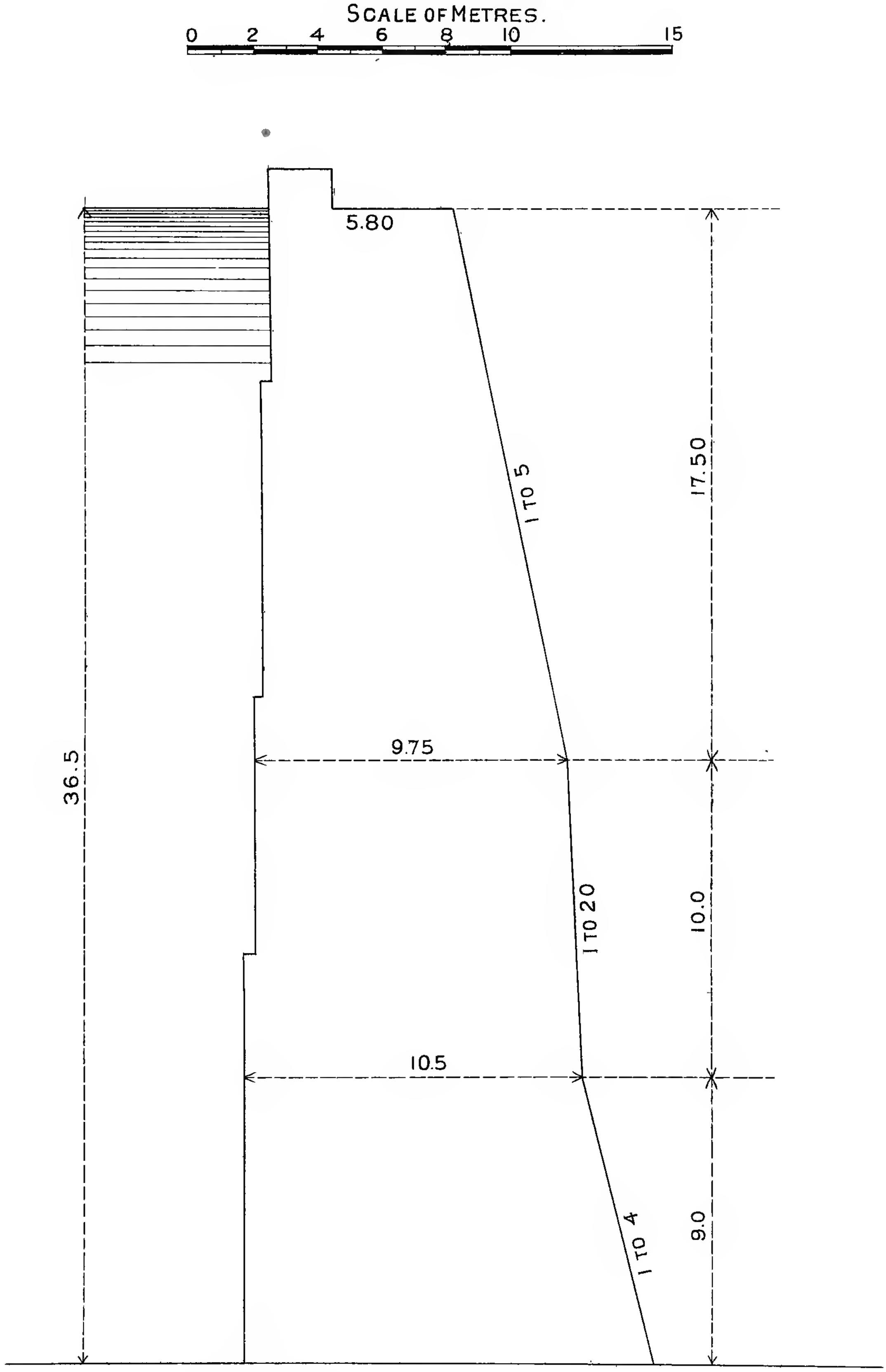
SEE PAGE 52

GROS-BOIS DAM



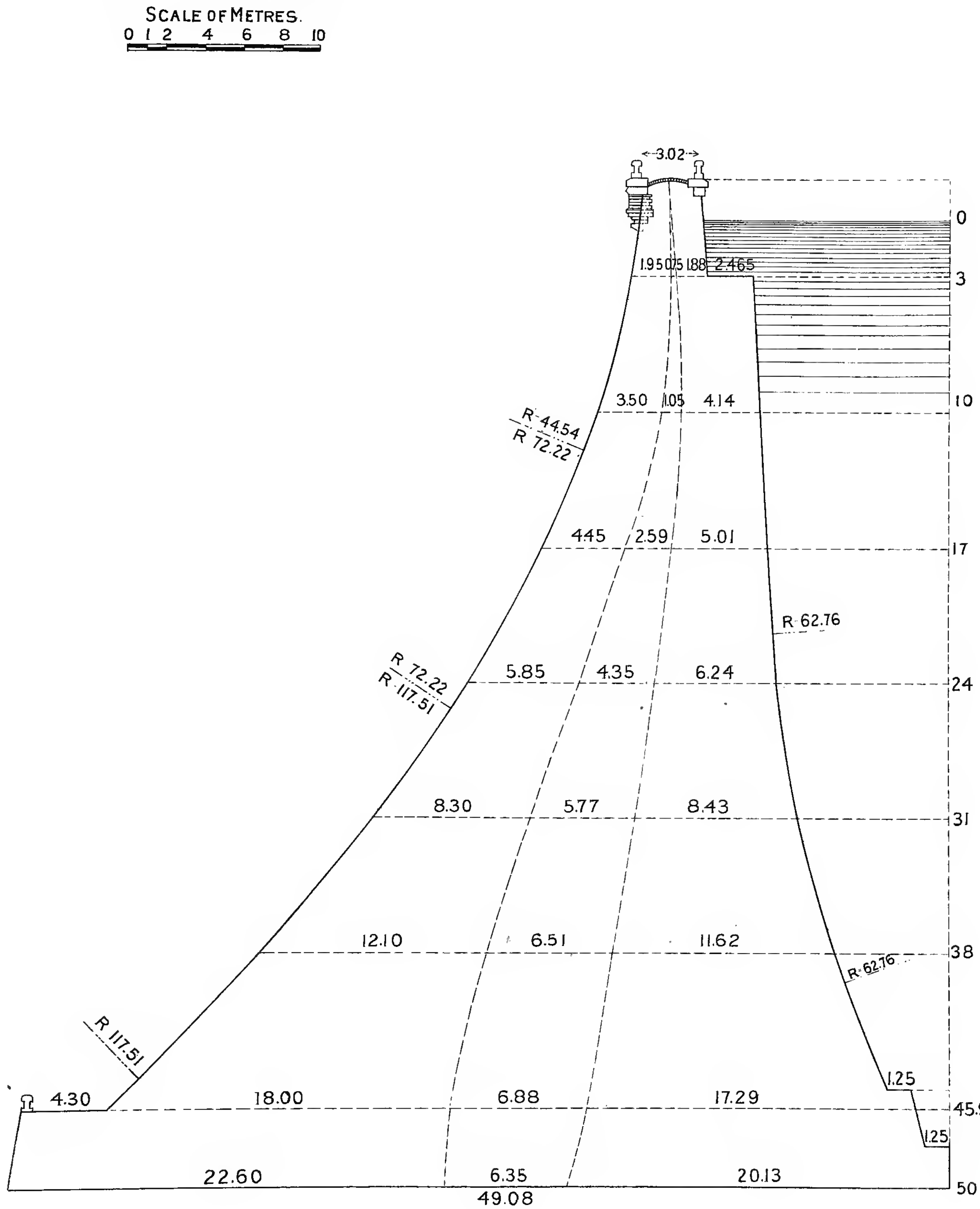
See Page 53

ZOLA DAM

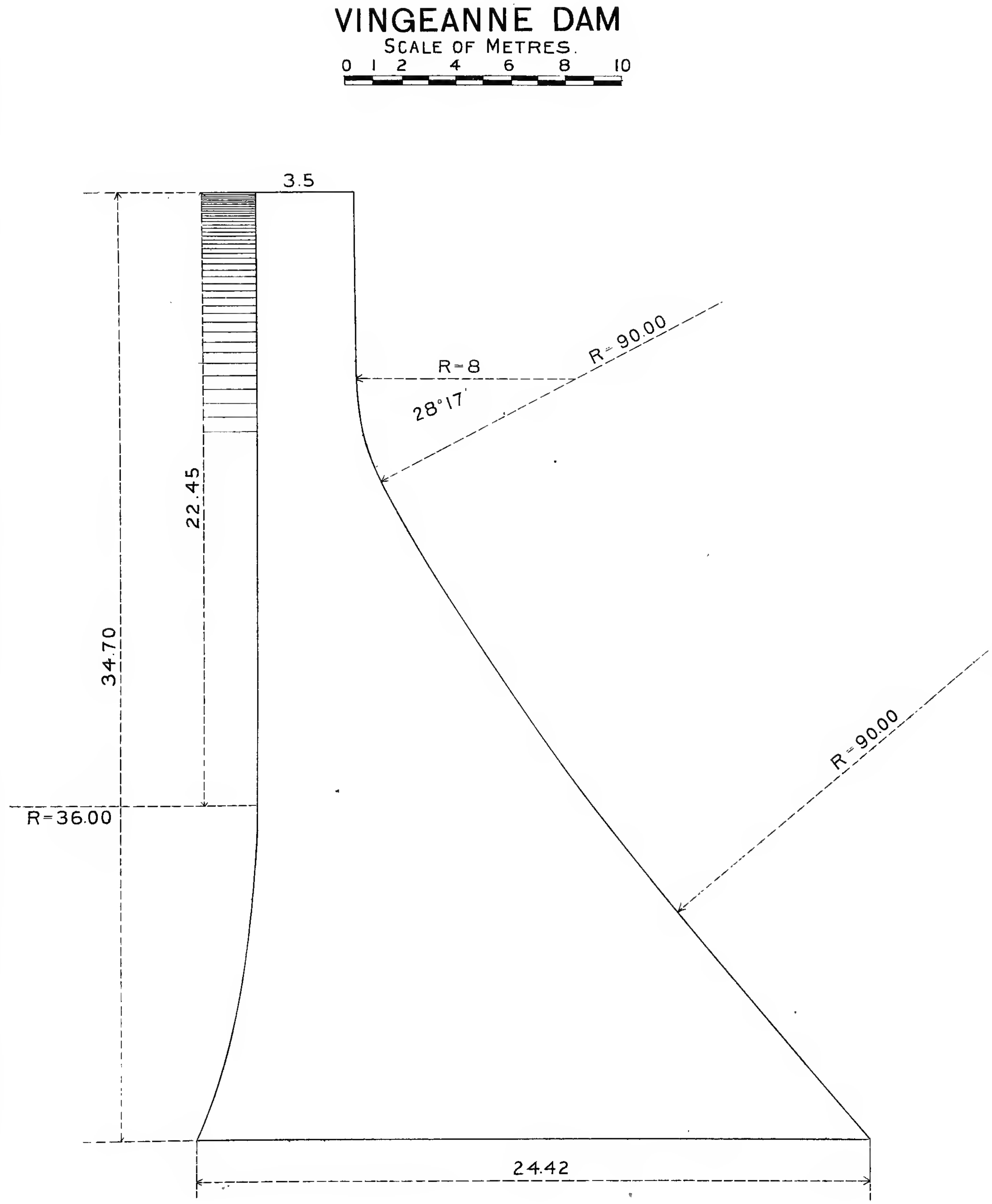


SEE PAGE 53

FUREN'S DAM

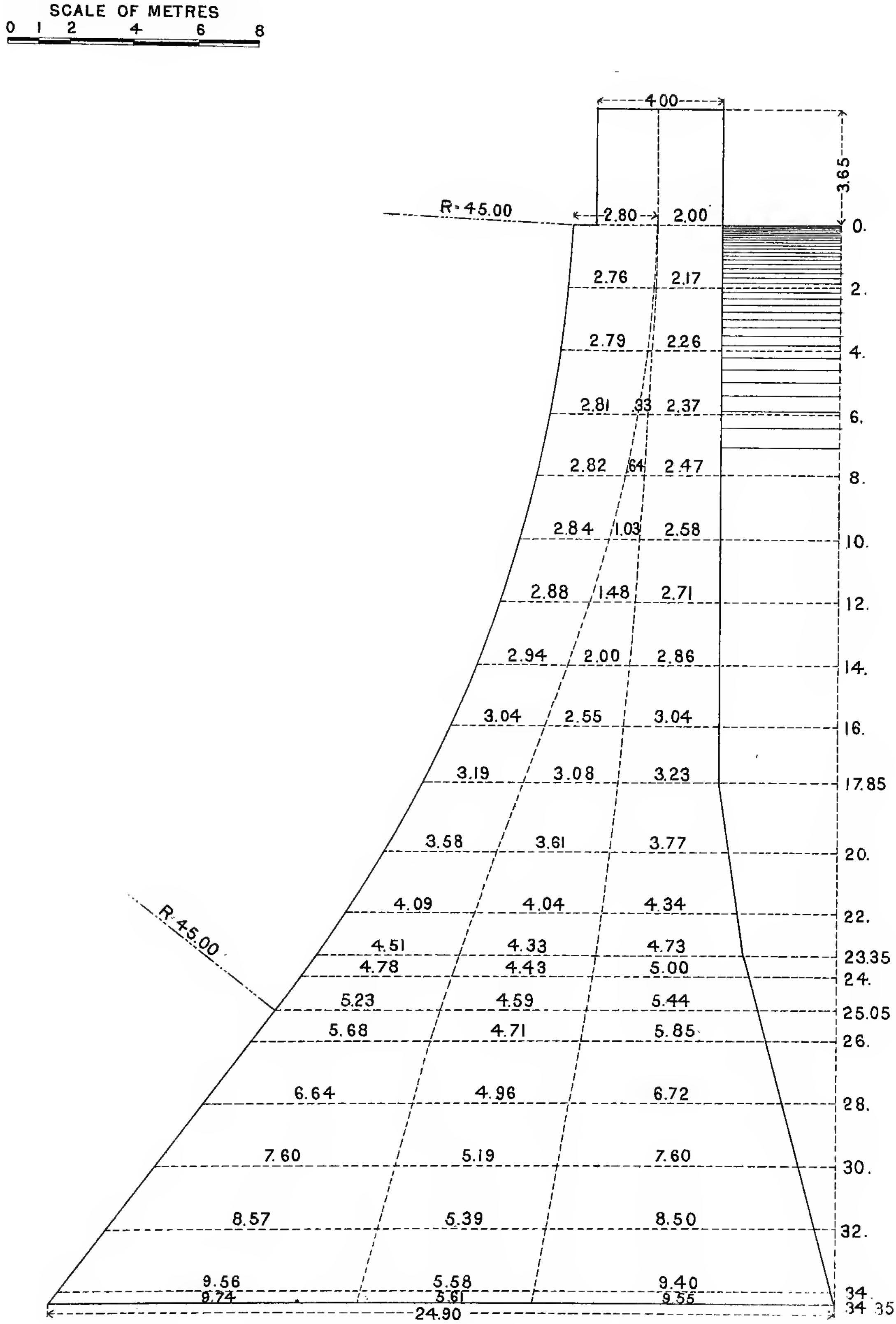


SEE PAGE 56


VINGEANNE DAM
SCALE OF METRES.
0 1 2 4 6 8 10
3.5
R=90.00
R=8
28°17'
22.45
34.70
R=90.00
R=36.00
24.42

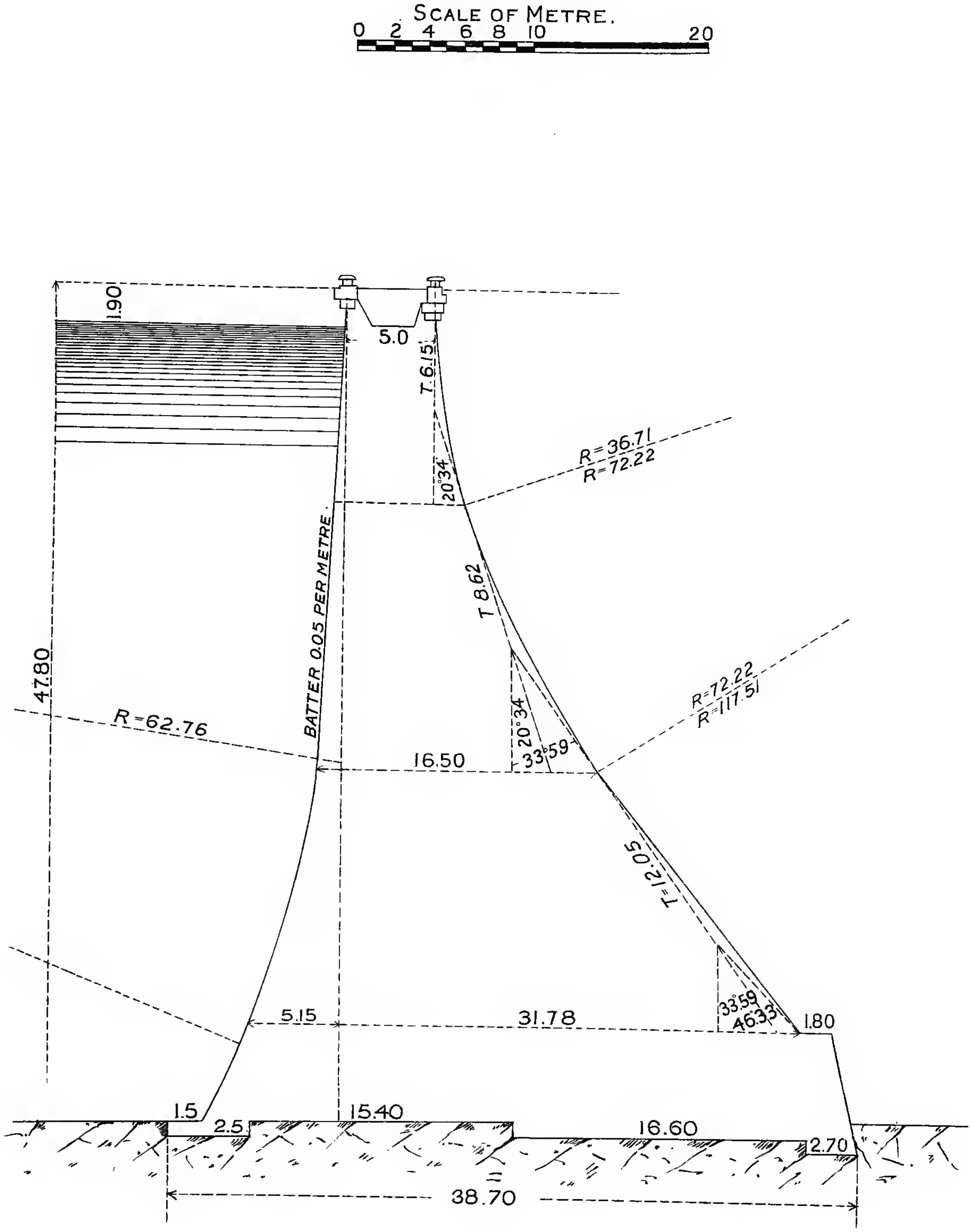
SEE PAGE 56

TERNAY DAM



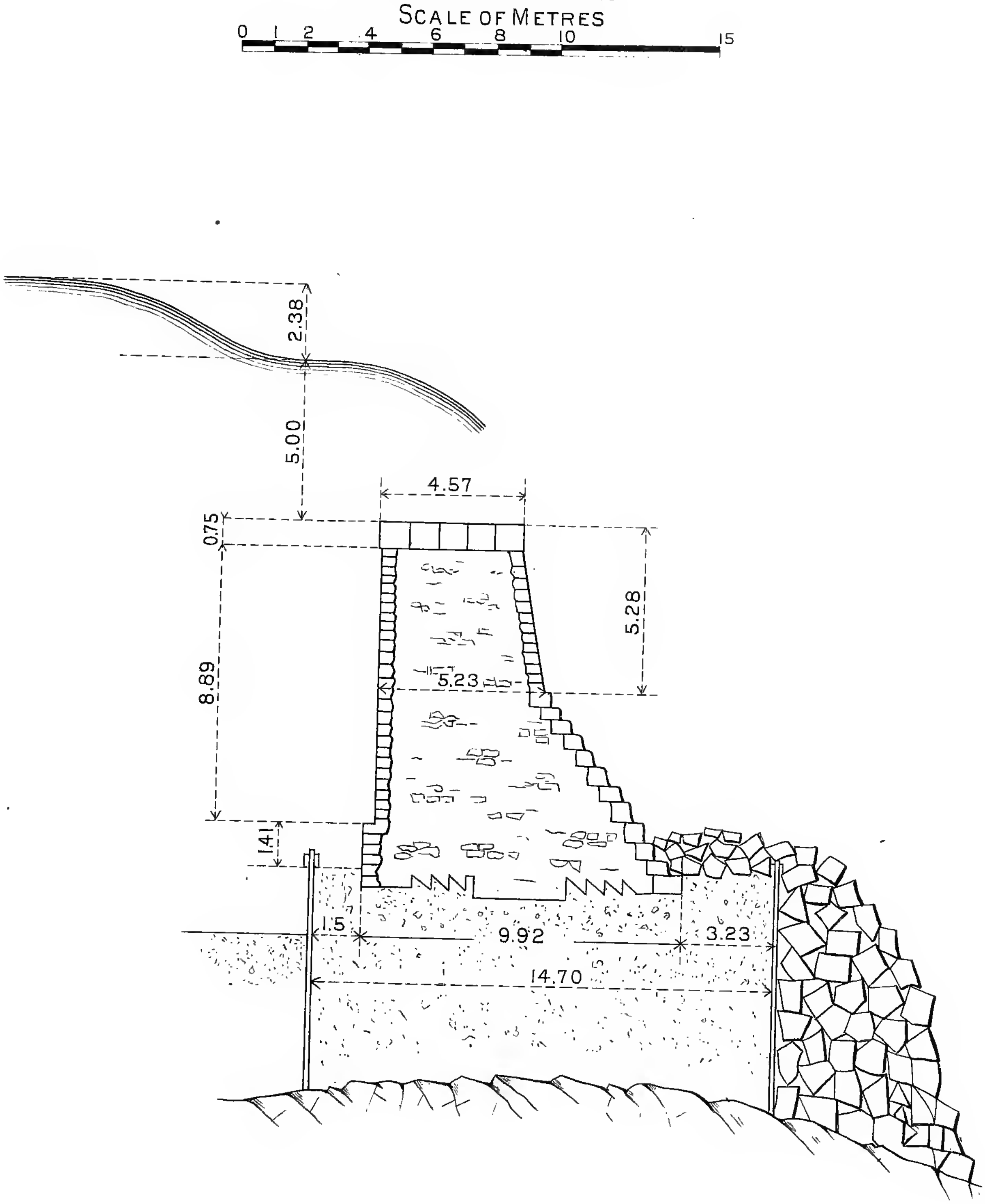
SEE PAGE 57

BAN DAM



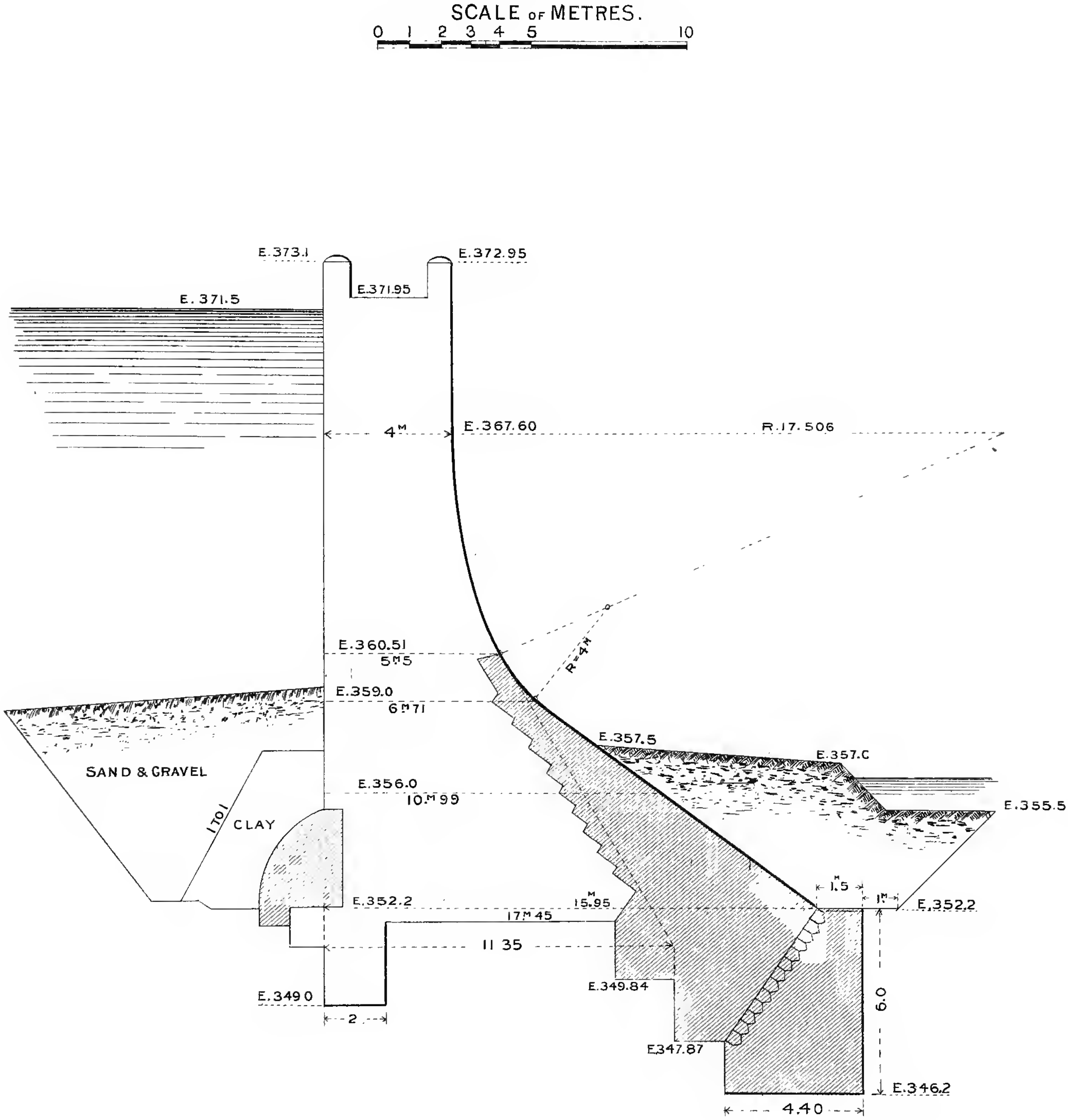
SEE PAGE 57

VERDON DAM

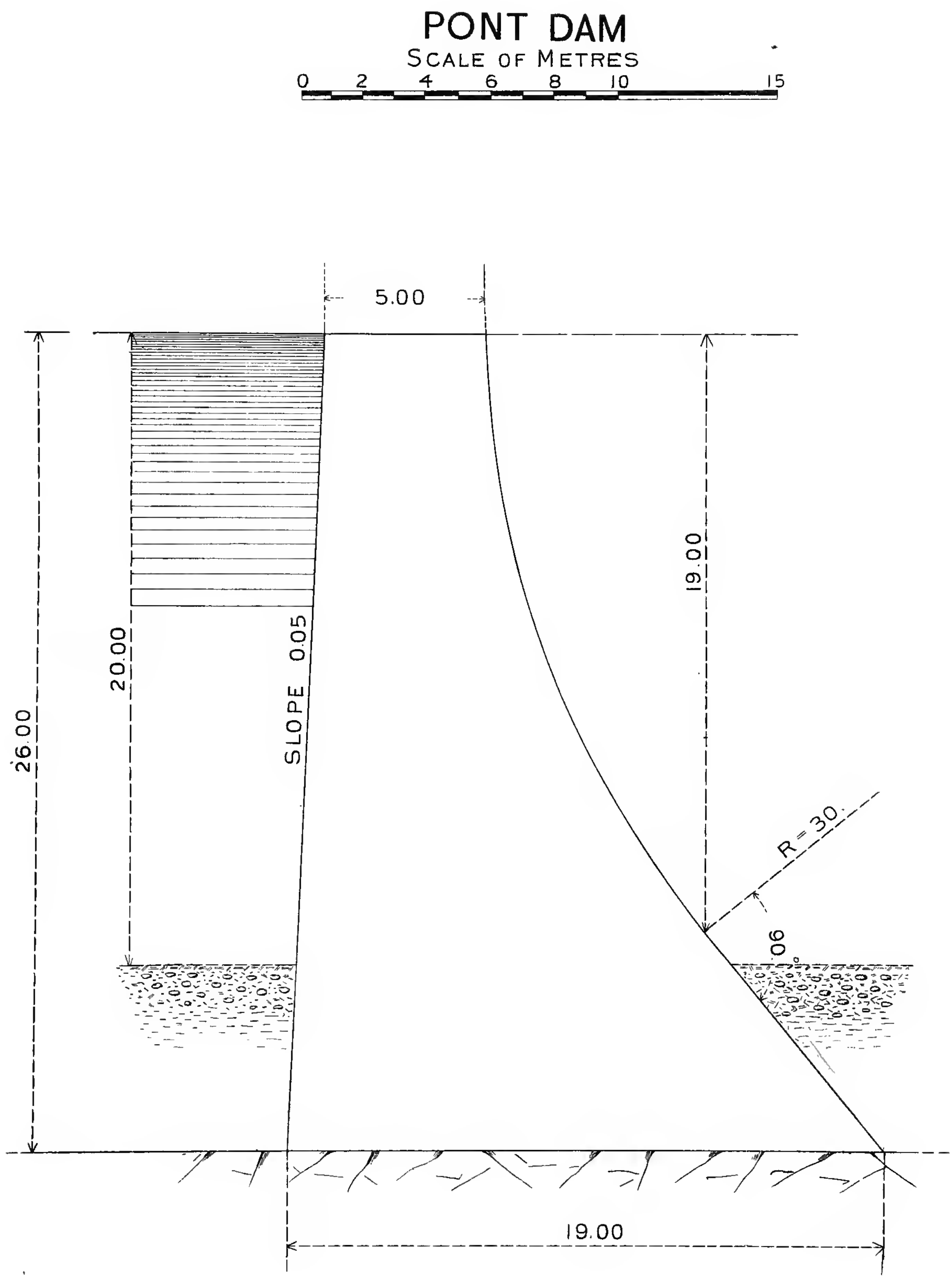


SEE PAGE 58

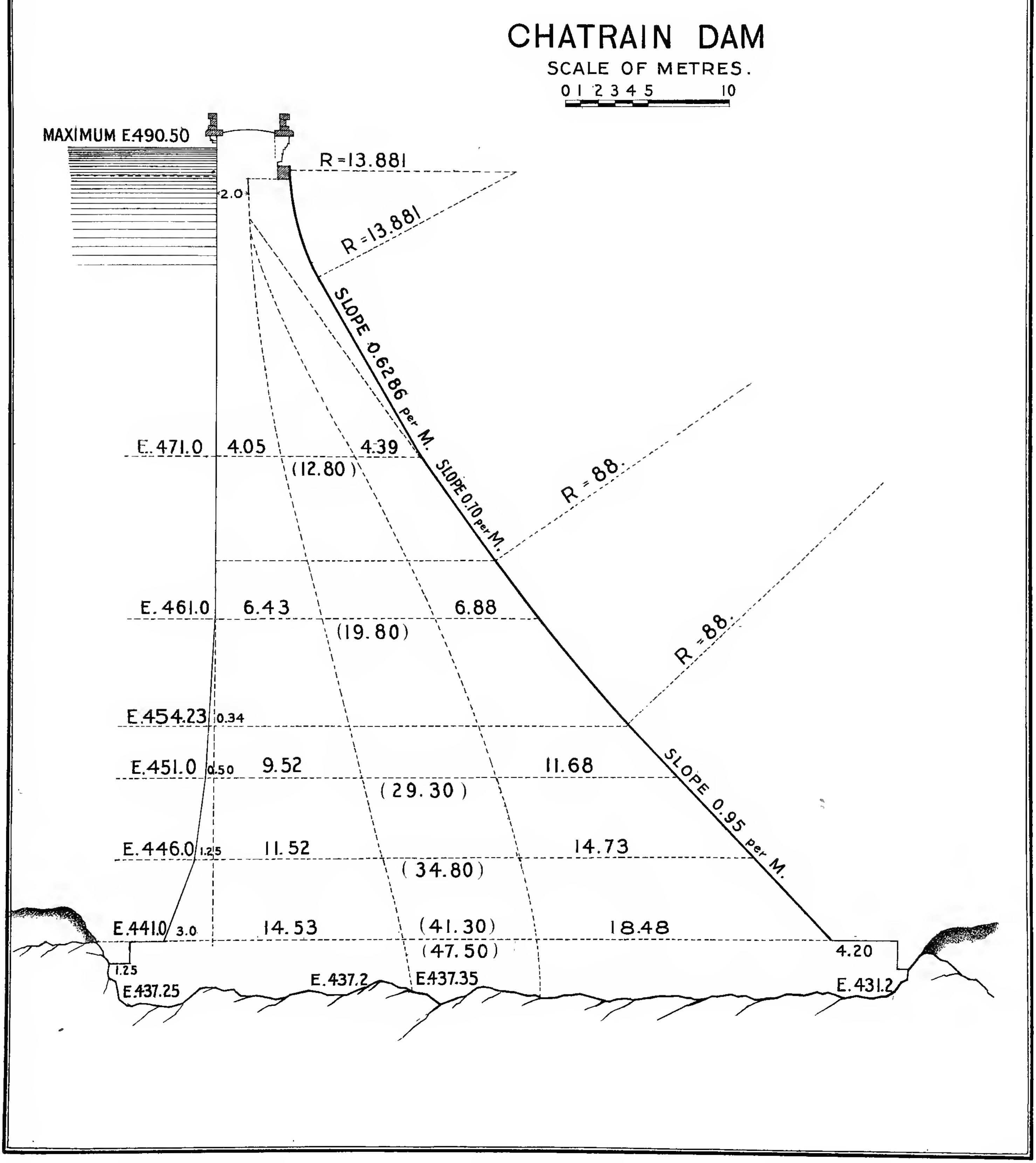
BOUZEY DAM



SEE PAGE 59

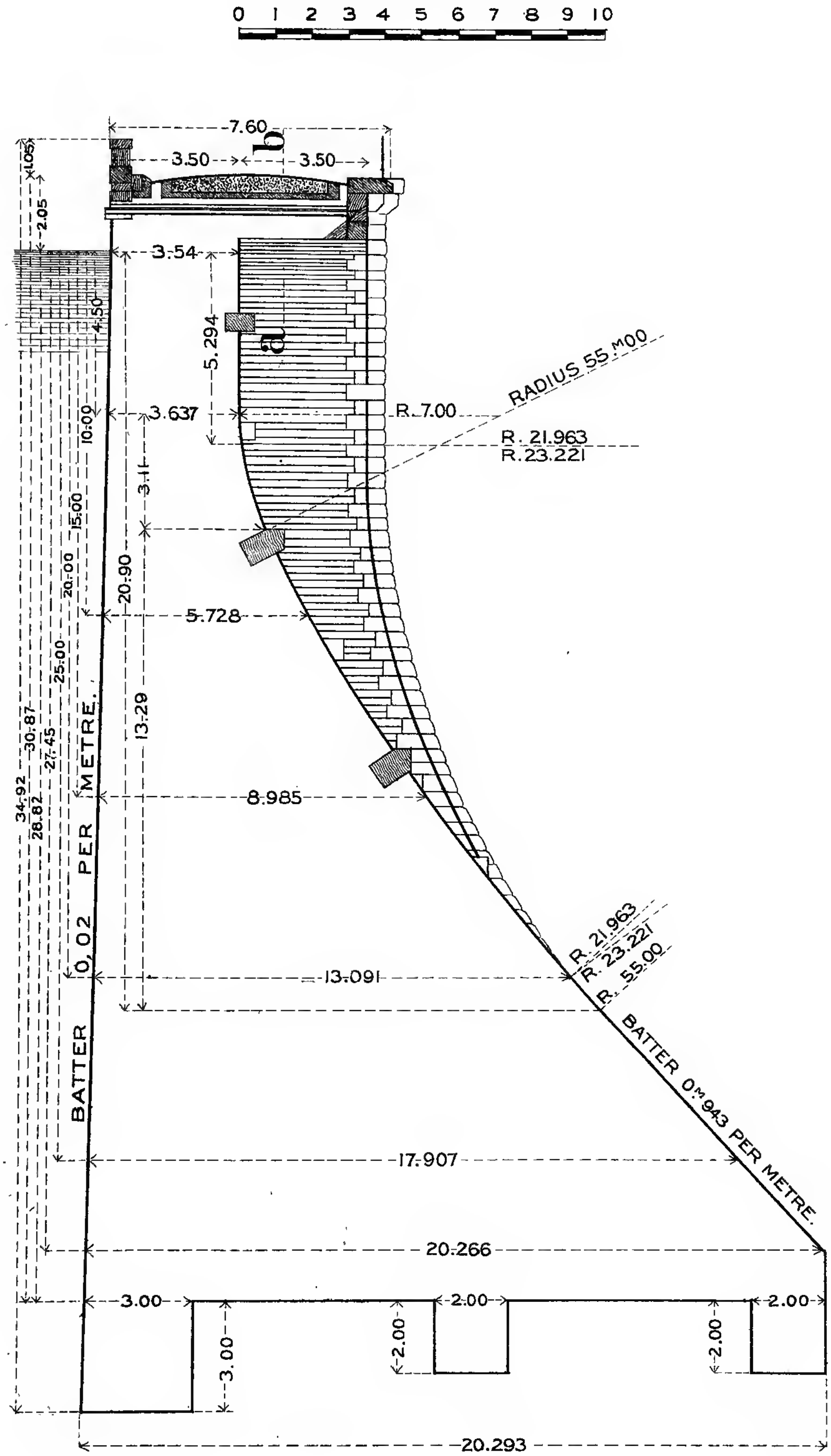

PONT DAM
SCALE OF METRES
0 2 4 6 8 10 15
5.00
26.00
20.00
SLOPE 0.05
19.00
R=30.
90°
19.00

SEE PAGE 59

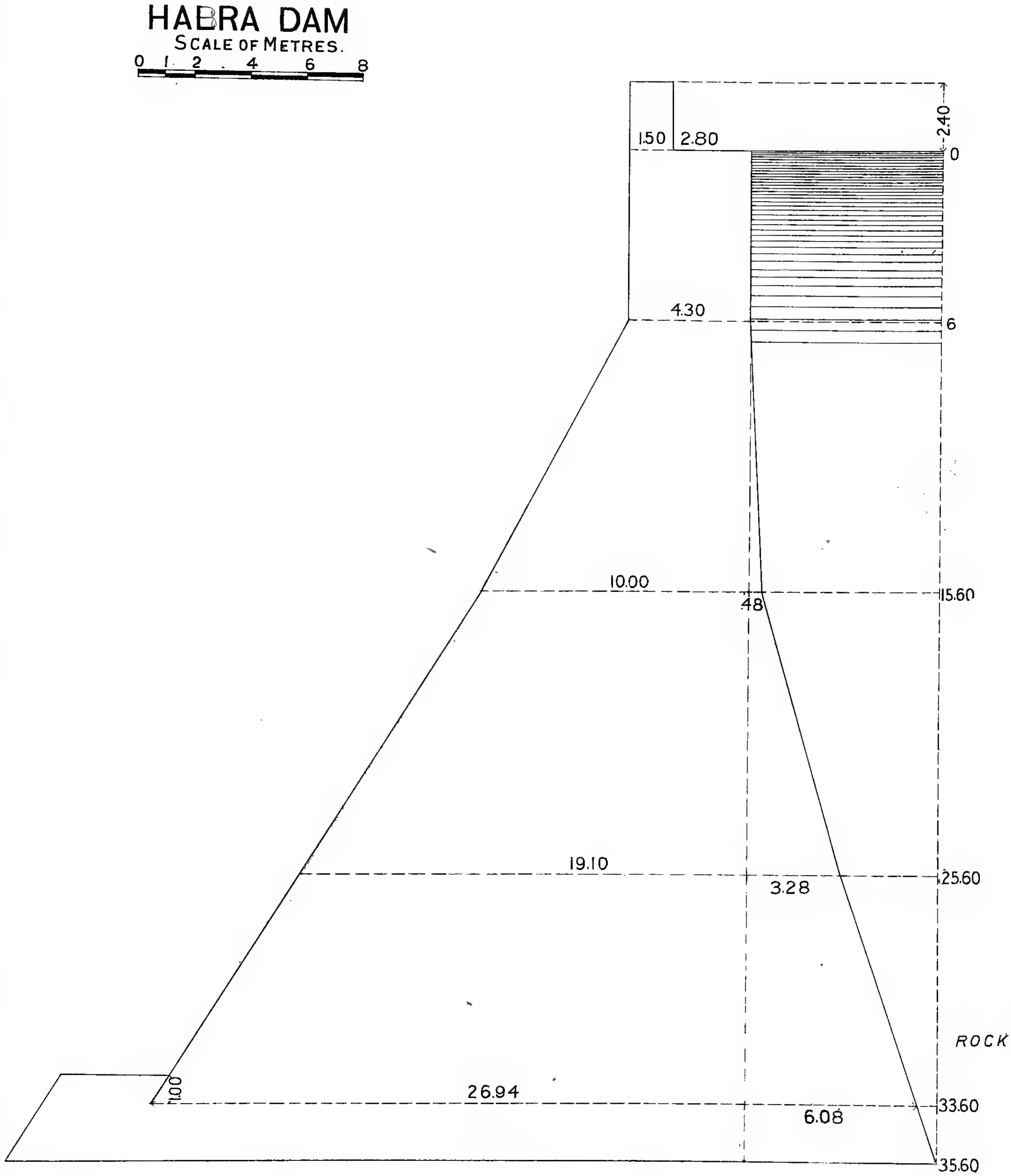

CHATRAIN DAM
SCALE OF METRES.
0 1 2 3 4 5 10
MAXIMUM E.490.50
R=13.881
R=13.881
2.0
SLOPE 0.6286 per M.
SLOPE 0.70 per M.
R=88.
R=88.
SLOPE 0.95 per M.
E.471.0 4.05 4.39
(12.80)
E.461.0 6.43 6.88
(19.80)
E.454.23 0.34
E.451.0 0.50 9.52 11.68
(29.30)
E.446.0 1.25 11.52 14.73
(34.80)
E.441.0 3.0 14.53 (41.30) 18.48
(47.50)
4.20
1.25
E.437.25
E.437.2
E.437.35
E.431.2

SEE PAGE 60

MOUCHE DAM

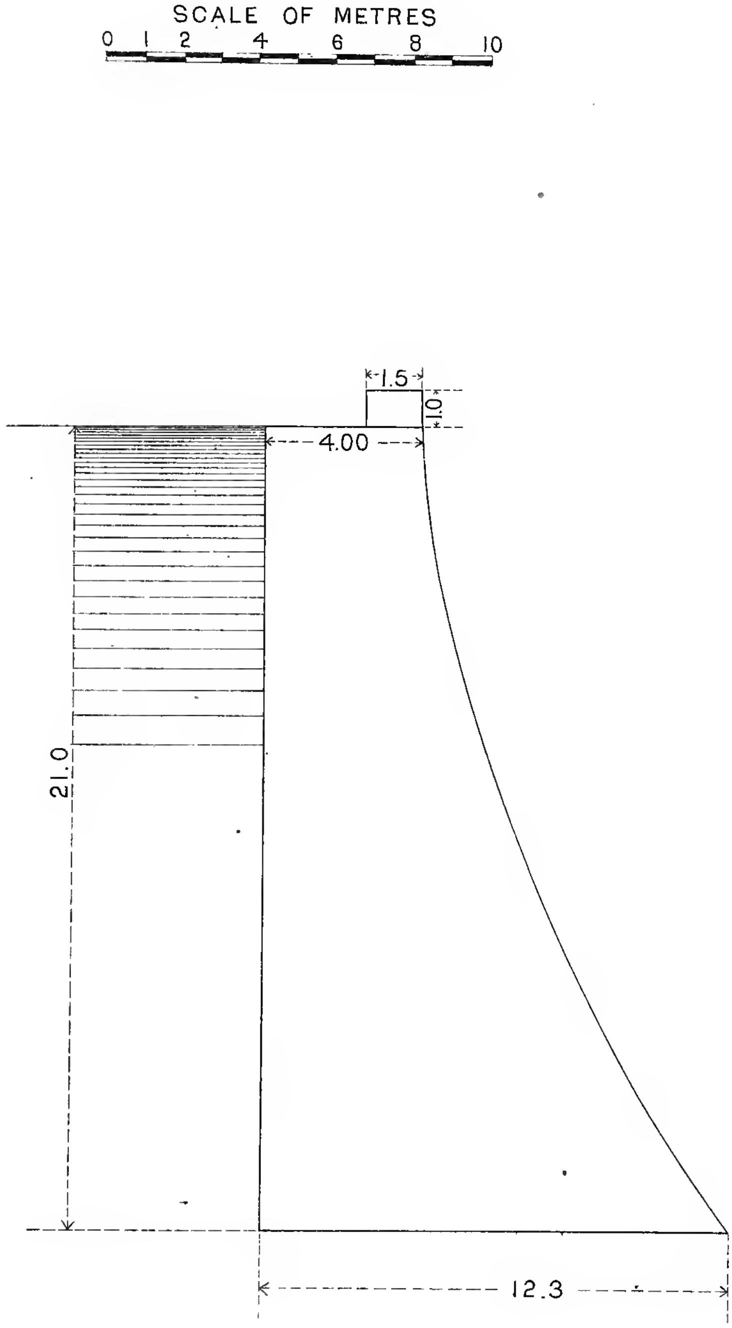


SEE PAGE 62


HABRA DAM
SCALE OF METRES.
0 1 2 4 6 8
1.50
2.80
2.40
0
4.30
6
10.00
.48
15.60
19.10
3.28
25.60
ROCK
1.00
26.94
33.60
6.08
35.60

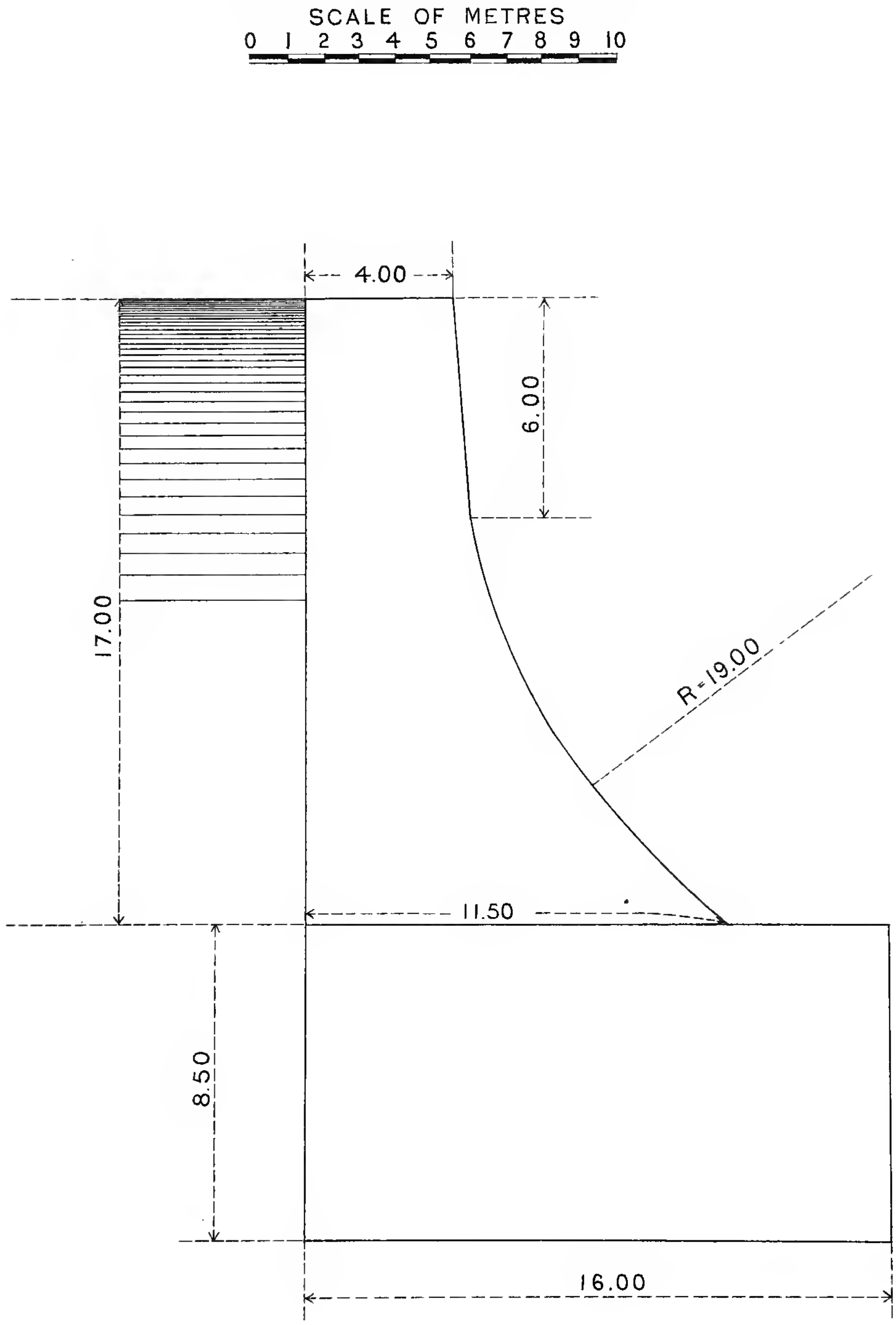
SEE PAGE 66

TLELAT DAM

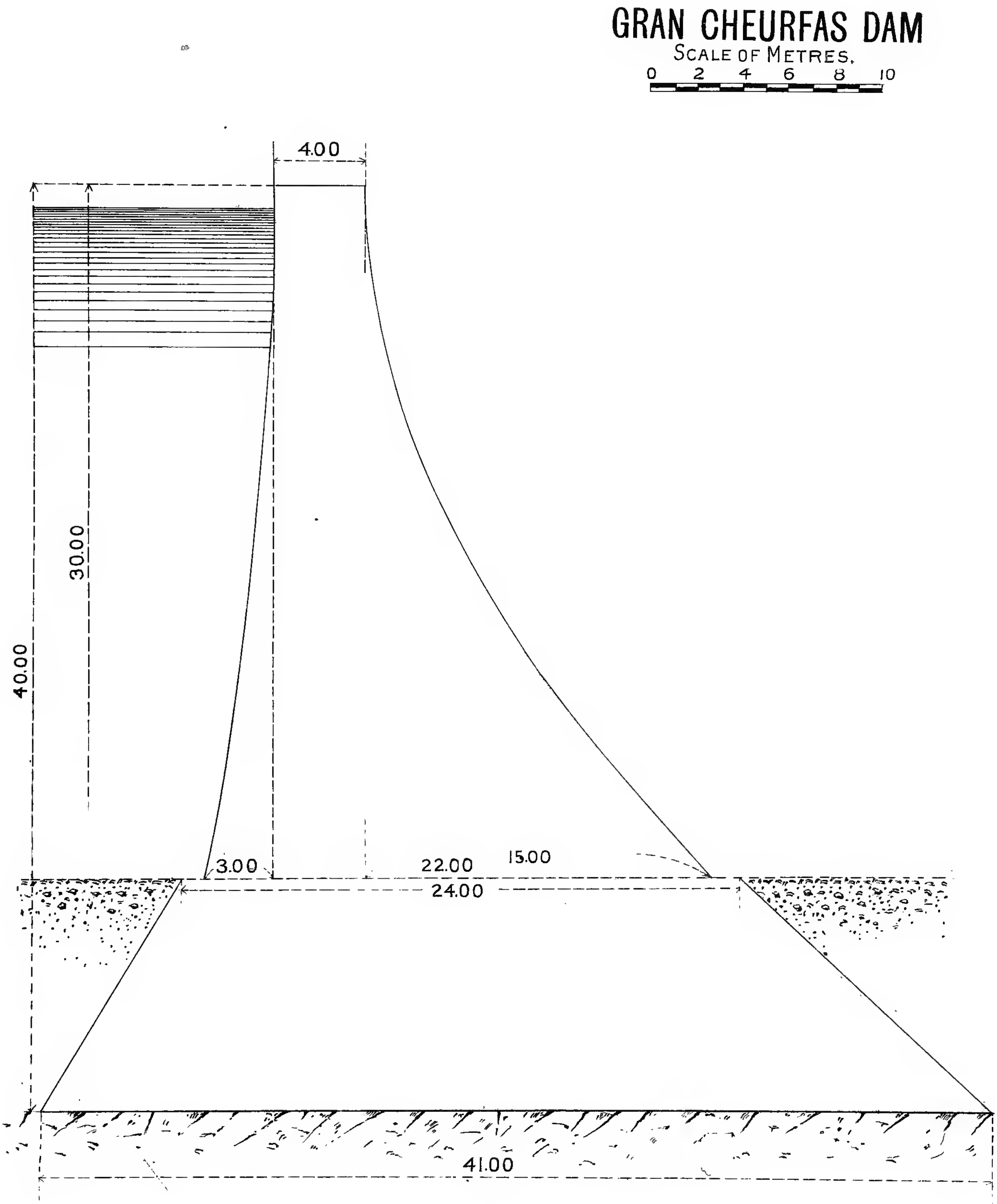


SEE PAGE 67

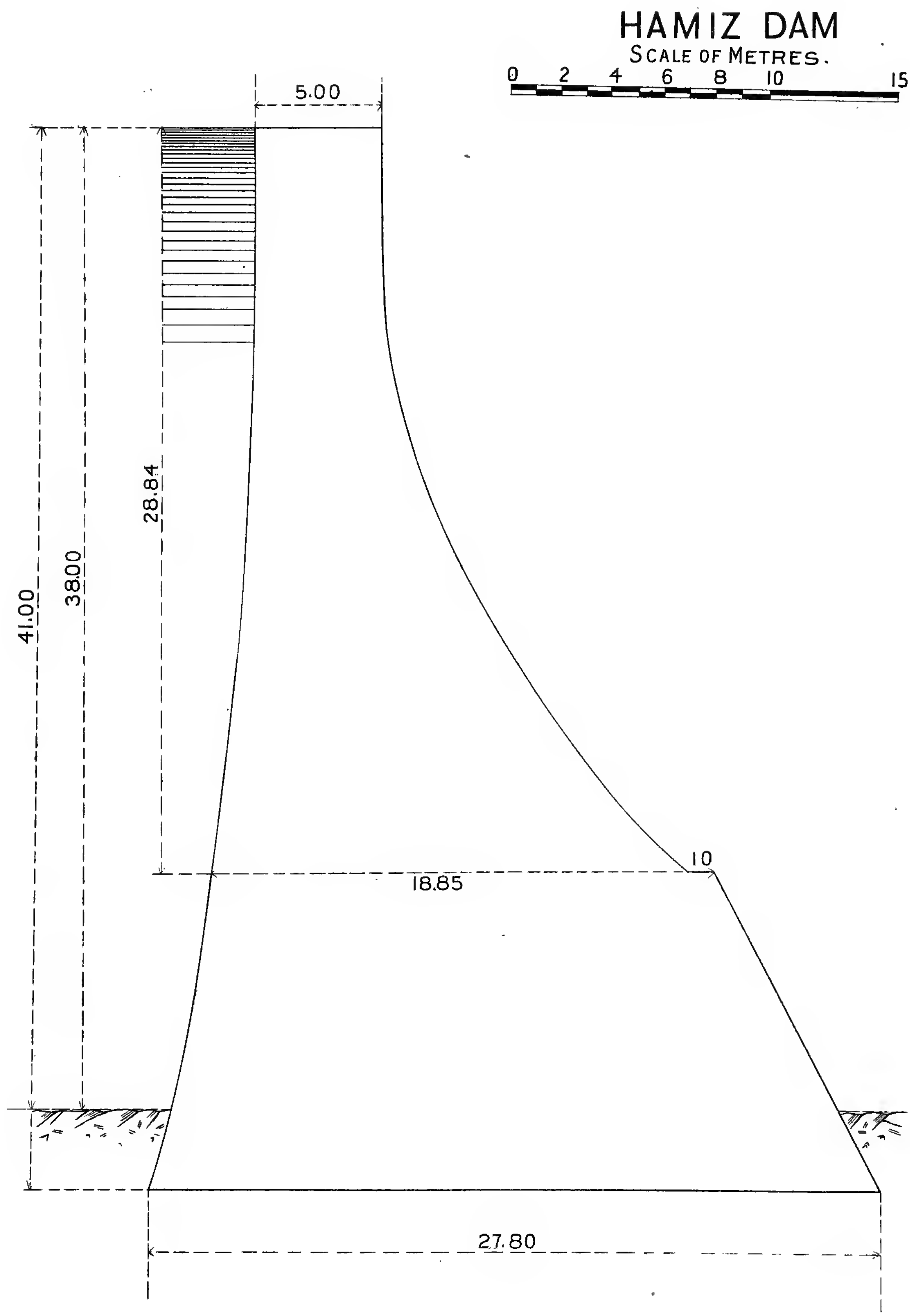
DJIDIONIA DAM



See Page 67

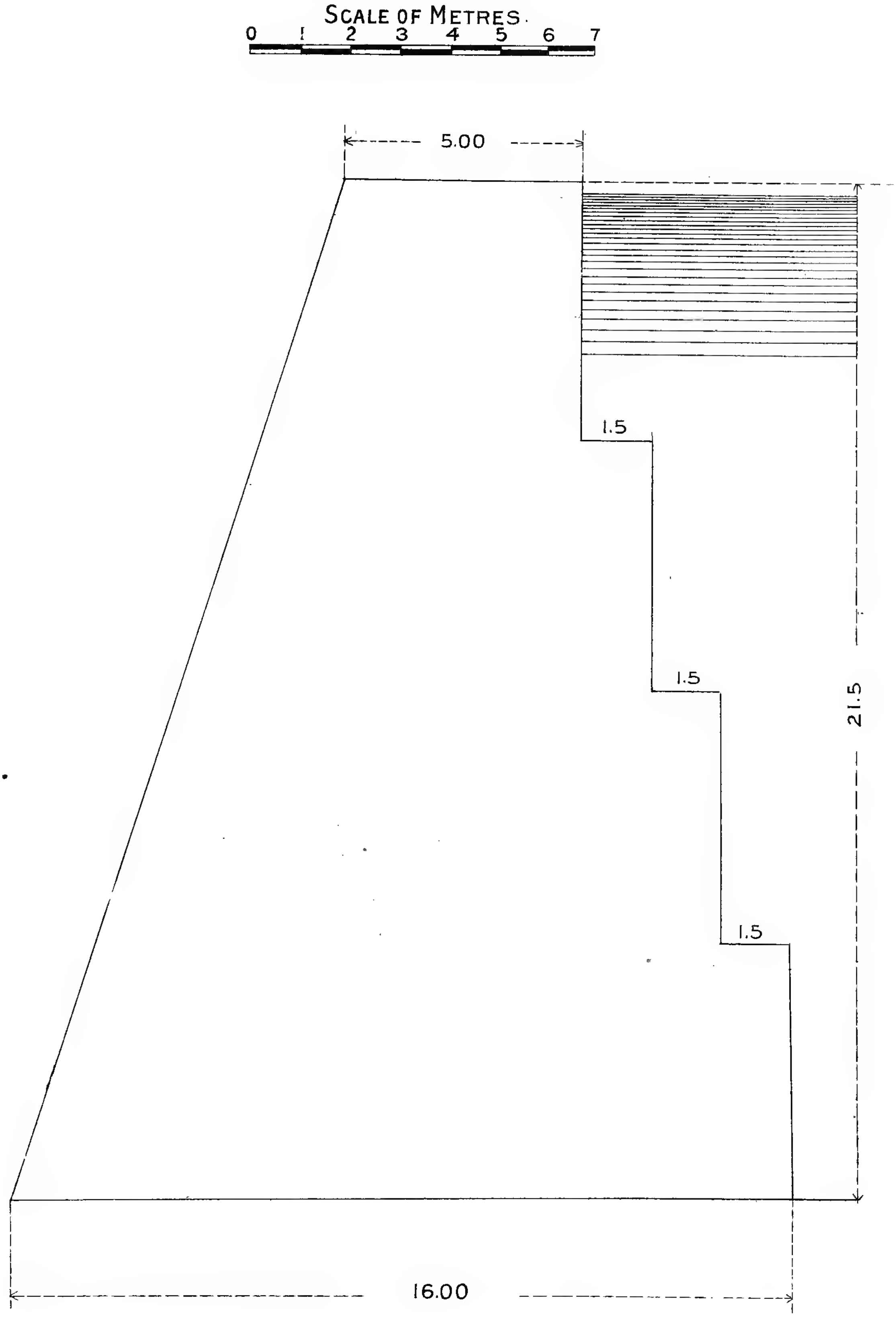

GRAN CHEURFAS DAM
Scale of Metres.
0 2 4 6 8 10
4.00
30.00
40.00
3.00
22.00
15.00
24.00
41.00

SEE PAGE 68.


HAMIZ DAM
SCALE OF METRES.
0 2 4 6 8 10 15
5.00
28.84
38.00
41.00
10
18.85
27.80

SEE PAGE 69.

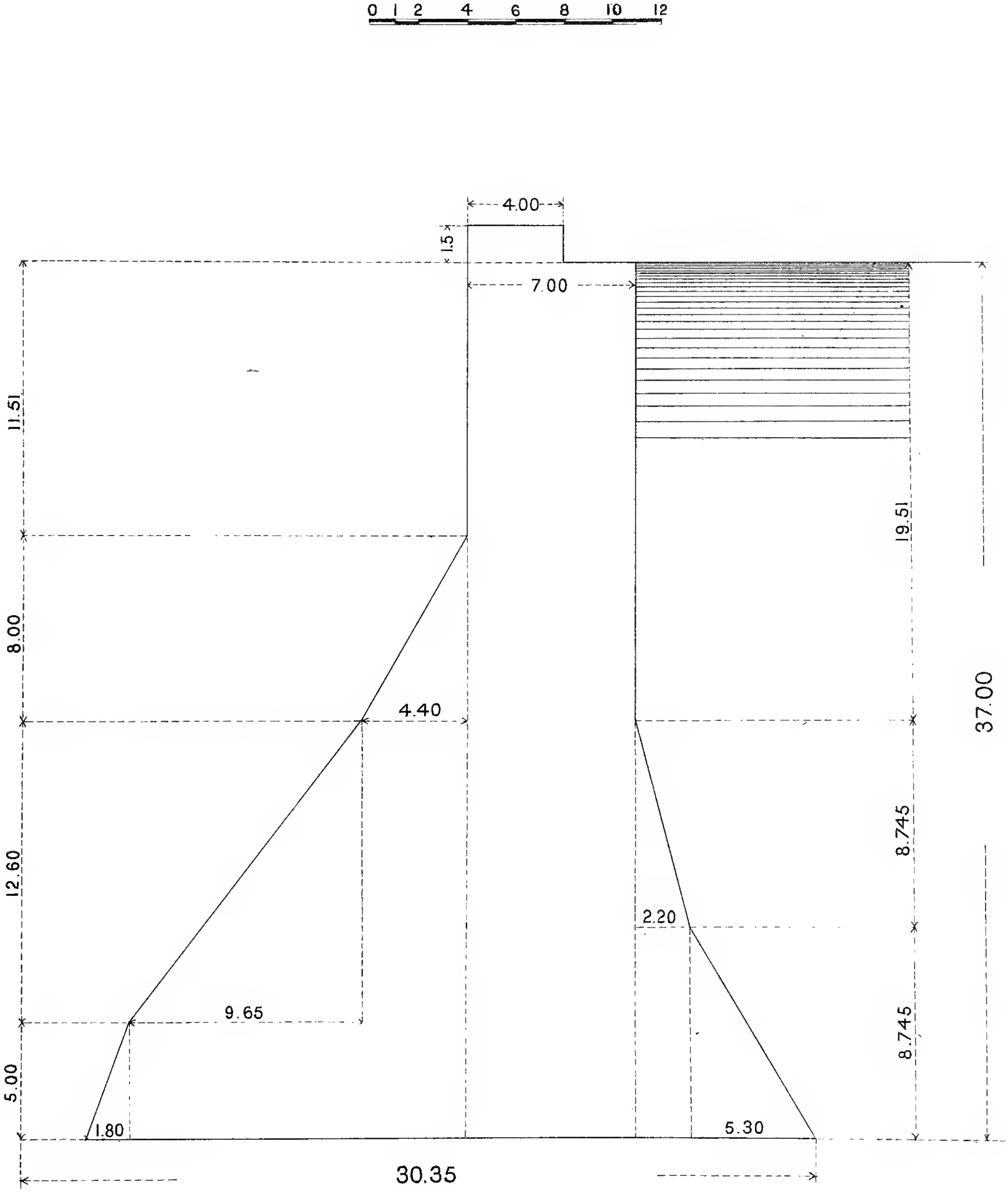
CAGLIARI DAM



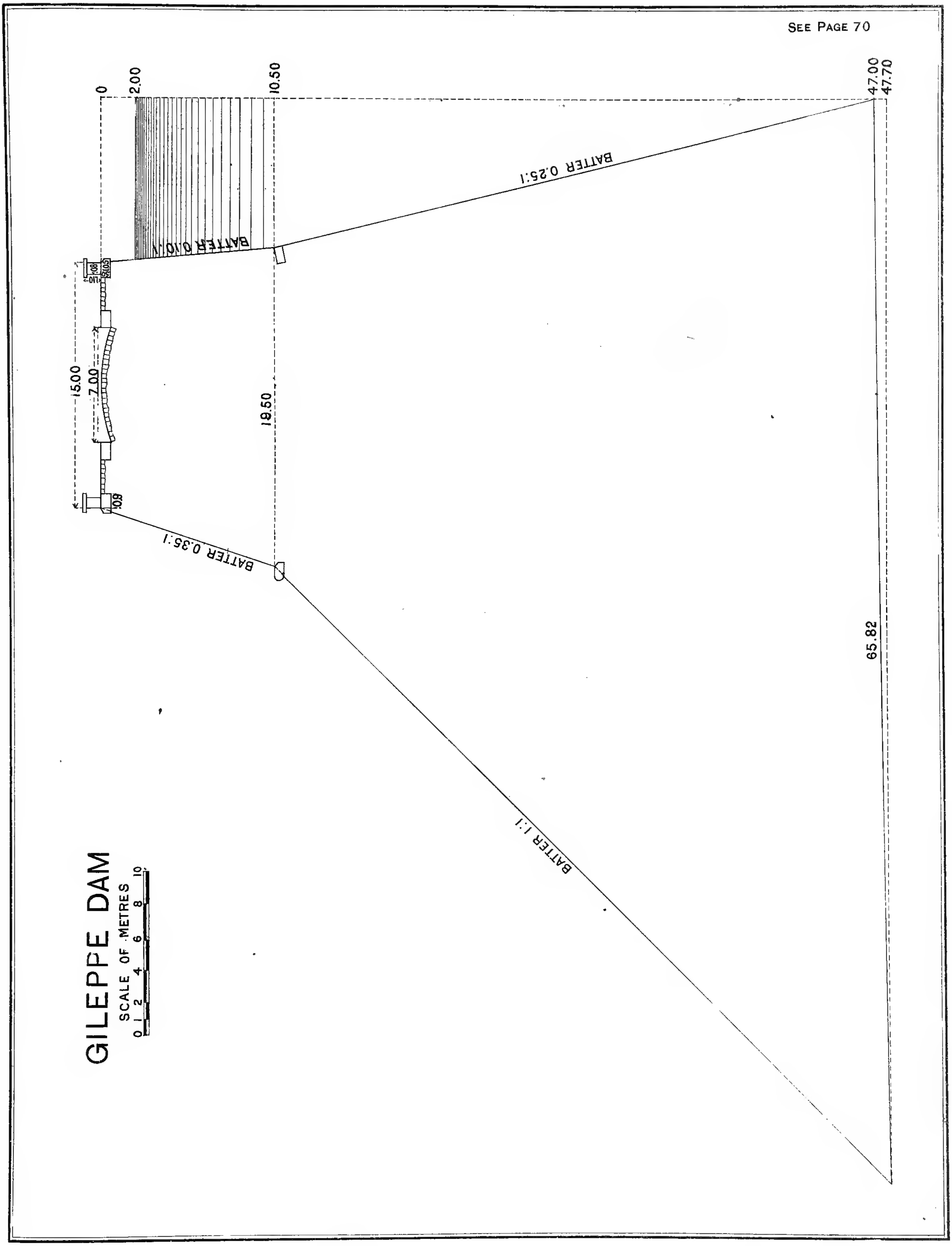
SEE PAGE 69

GORZENTE DAM

SCALE OF METRES

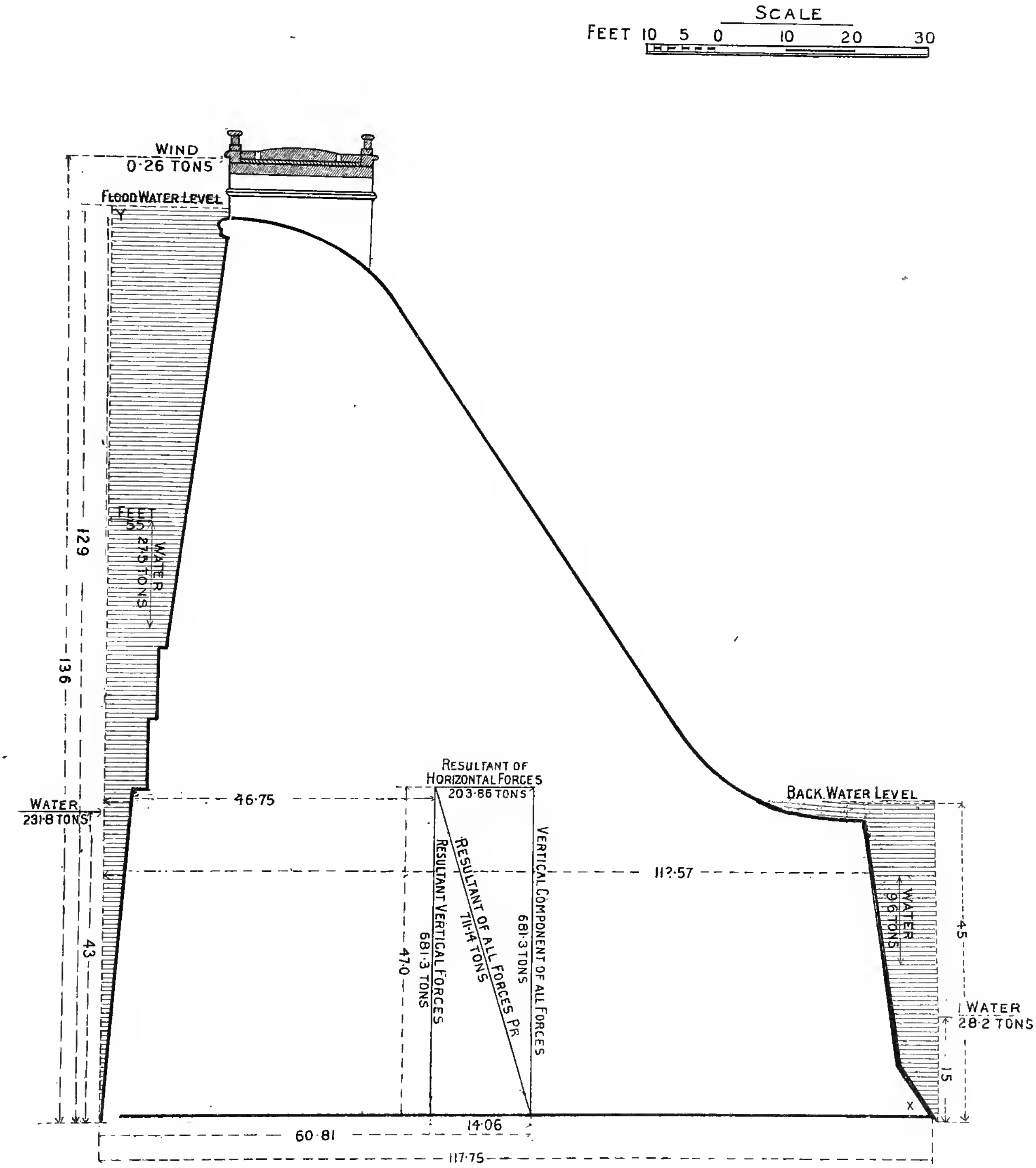


SEE PAGE 70


GILEPPE DAM
SCALE OF METRES
0 1 2 4 6 8 10
0
2.00
10.50
47.00
47.70
BATTER 0.25:1
BATTER 0.10:1
BATTER 0.35:1
BATTER 1:1
15.00
7.00
19.50
65.82

SEE PAGE 71

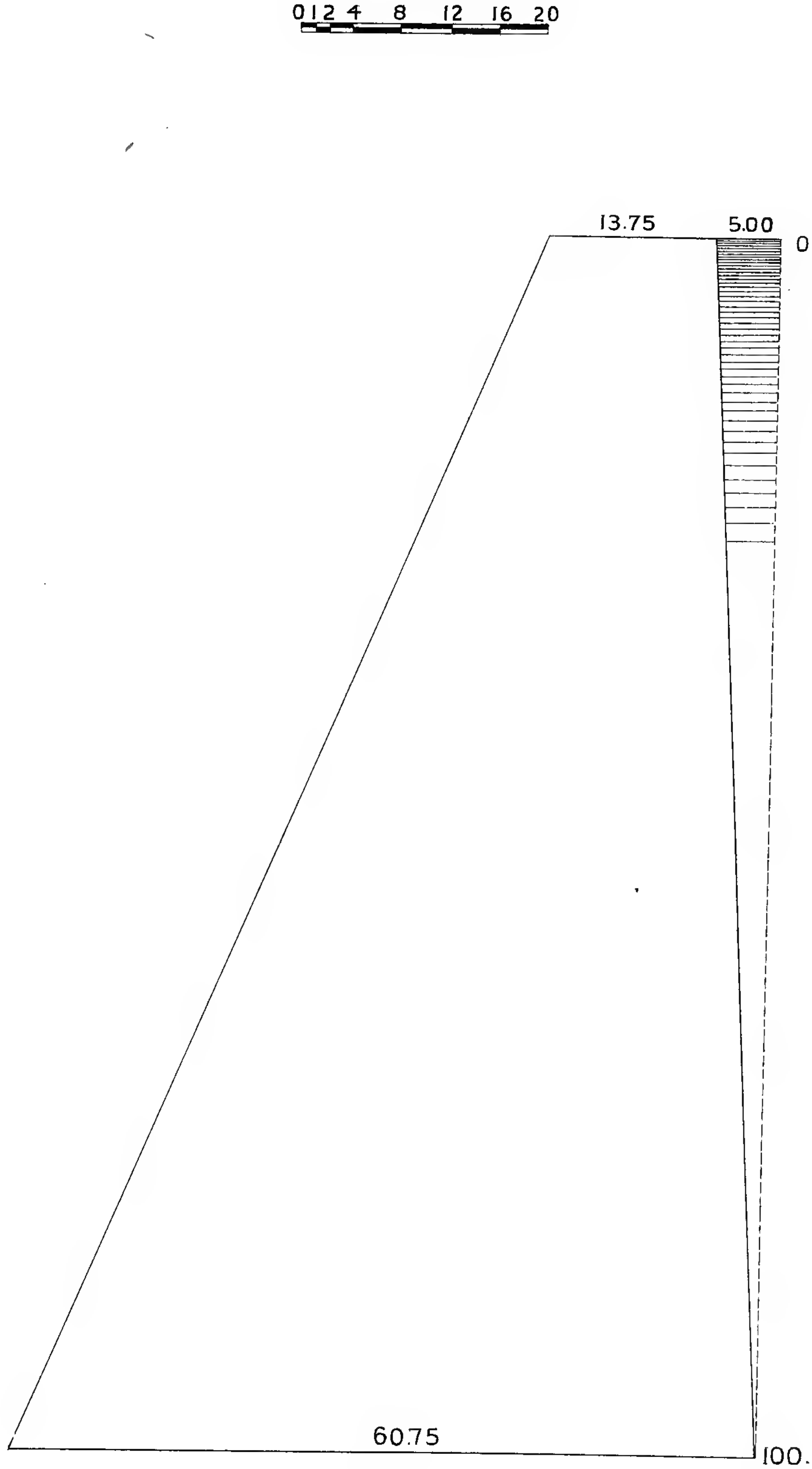
VYRNWY DAM



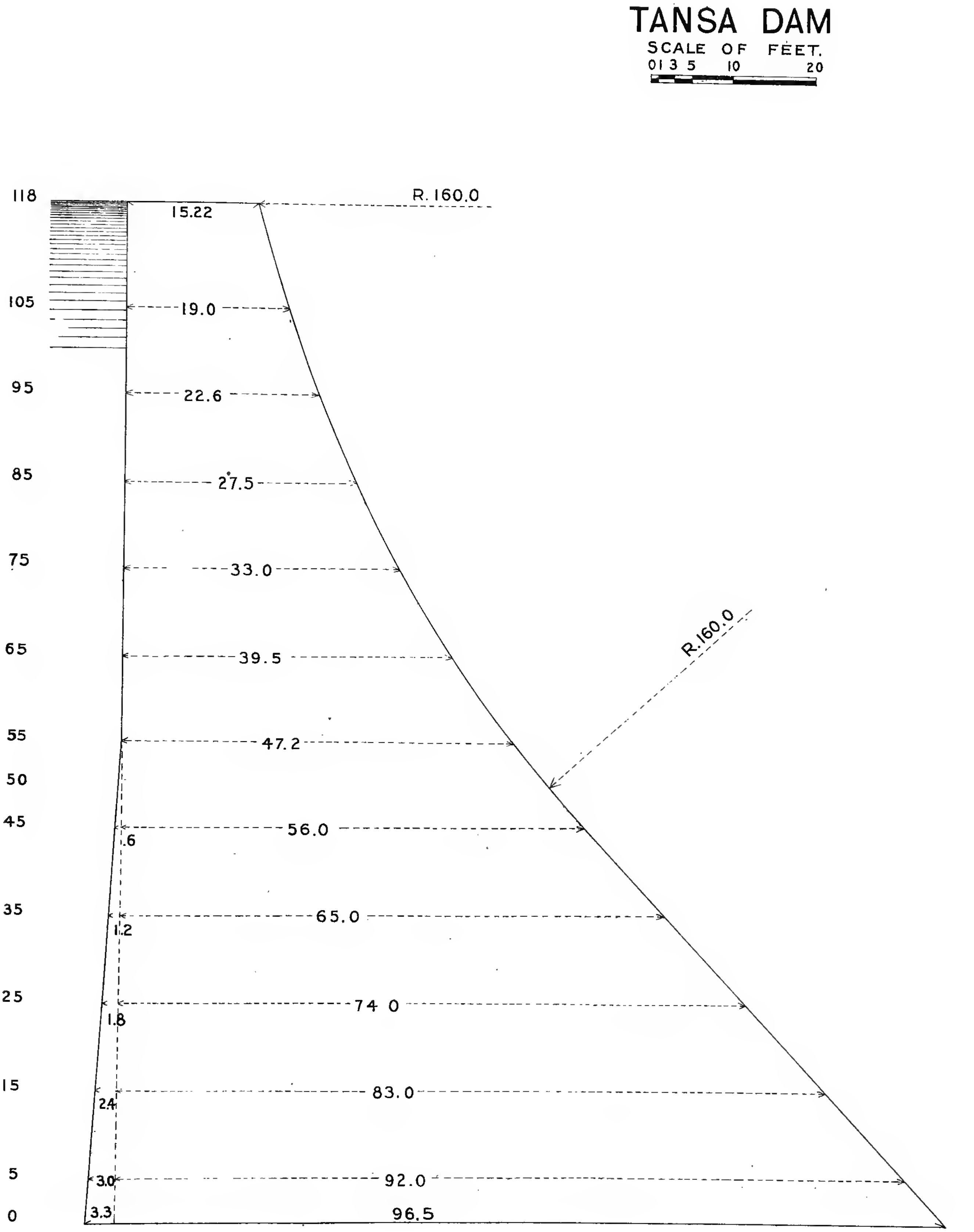
SEE PAGE 75

POONA DAM

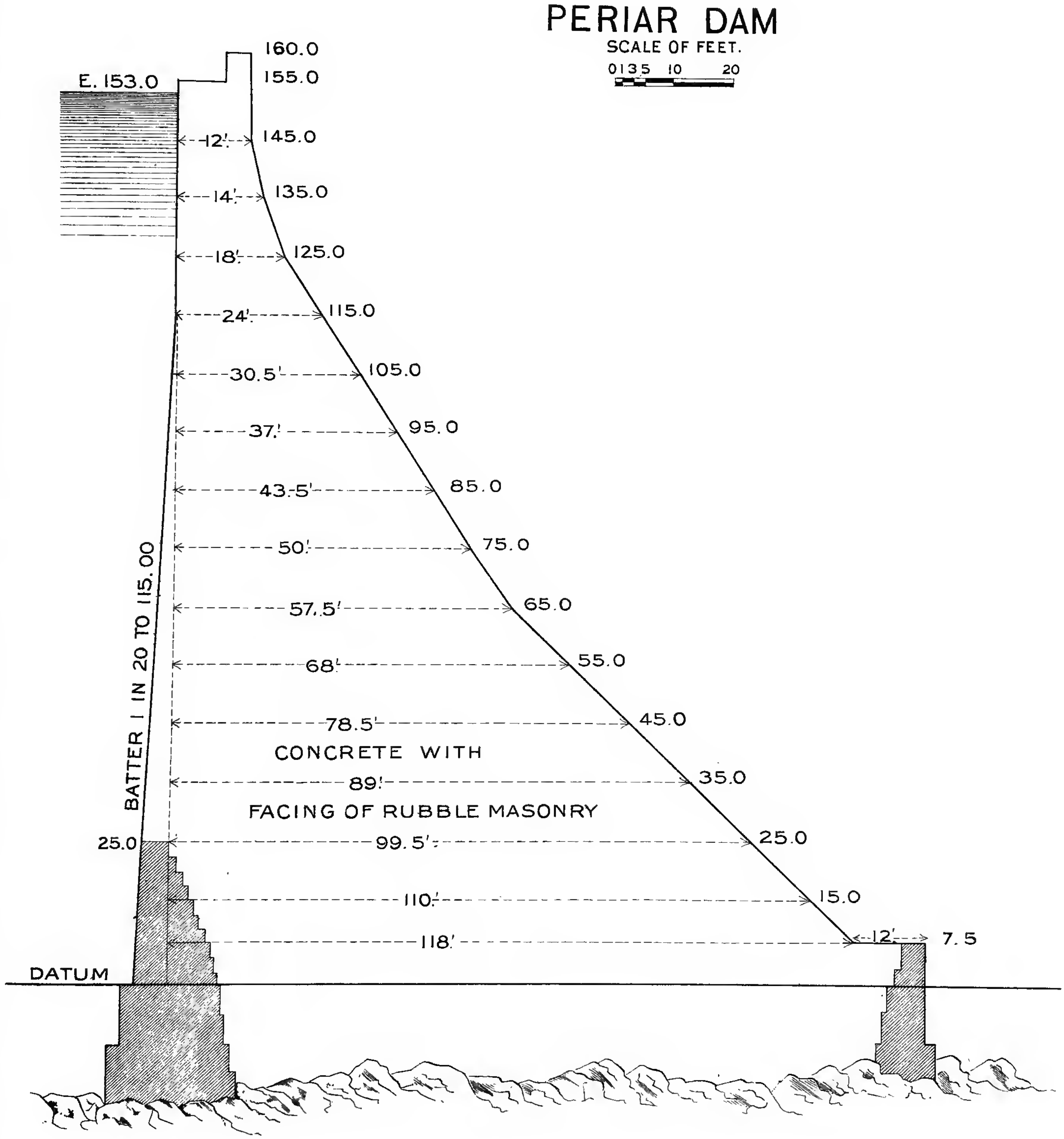
SCALE OF FEET



SEE PAGE 75

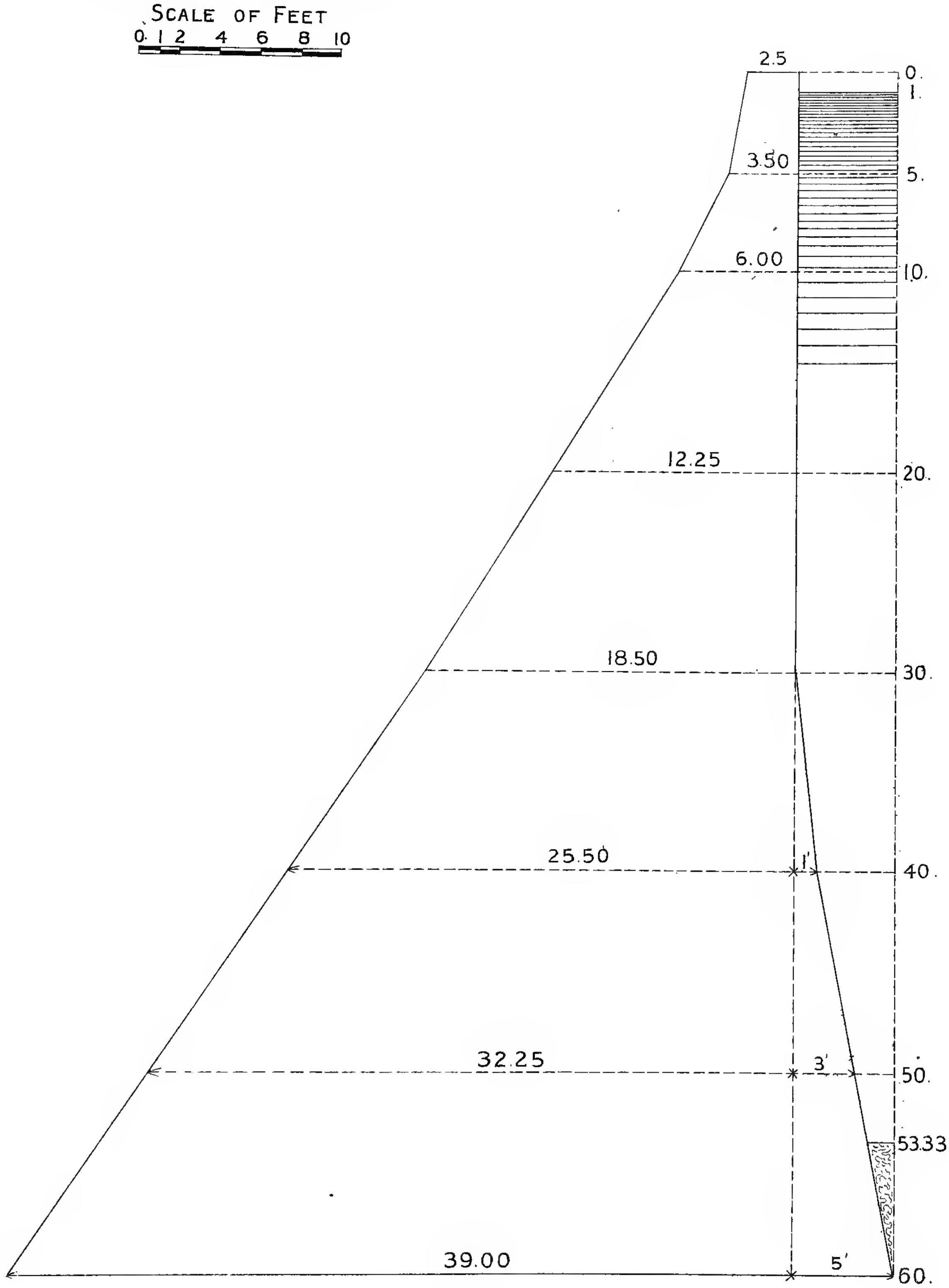

TANSA DAM
SCALE OF FEET.
0 1 3 5 10 20
118
R. 160.0
15.22
105
19.0
95
22.6
85
27.5
75
33.0
65
39.5
55
47.2
50
45
56.0
.6
35
65.0
1.2
25
74 0
1.8
15
83.0
2.4
5
92.0
3.0
0
3.3
96.5
R. 160.0

See Page 79


PERIAR DAM
SCALE OF FEET.
0 1 3 5 10 20
160.0
E. 153.0
155.0
12'
145.0
14'
135.0
18'
125.0
24'
115.0
30.5'
105.0
37'
95.0
43.5'
85.0
50'
75.0
57.5'
65.0
68'
55.0
78.5'
45.0
CONCRETE WITH
89'
35.0
FACING OF RUBBLE MASONRY
BATTER 1 IN 20 TO 115.00
25.0
99.5'
25.0
110'
15.0
118'
12'
7.5
DATUM

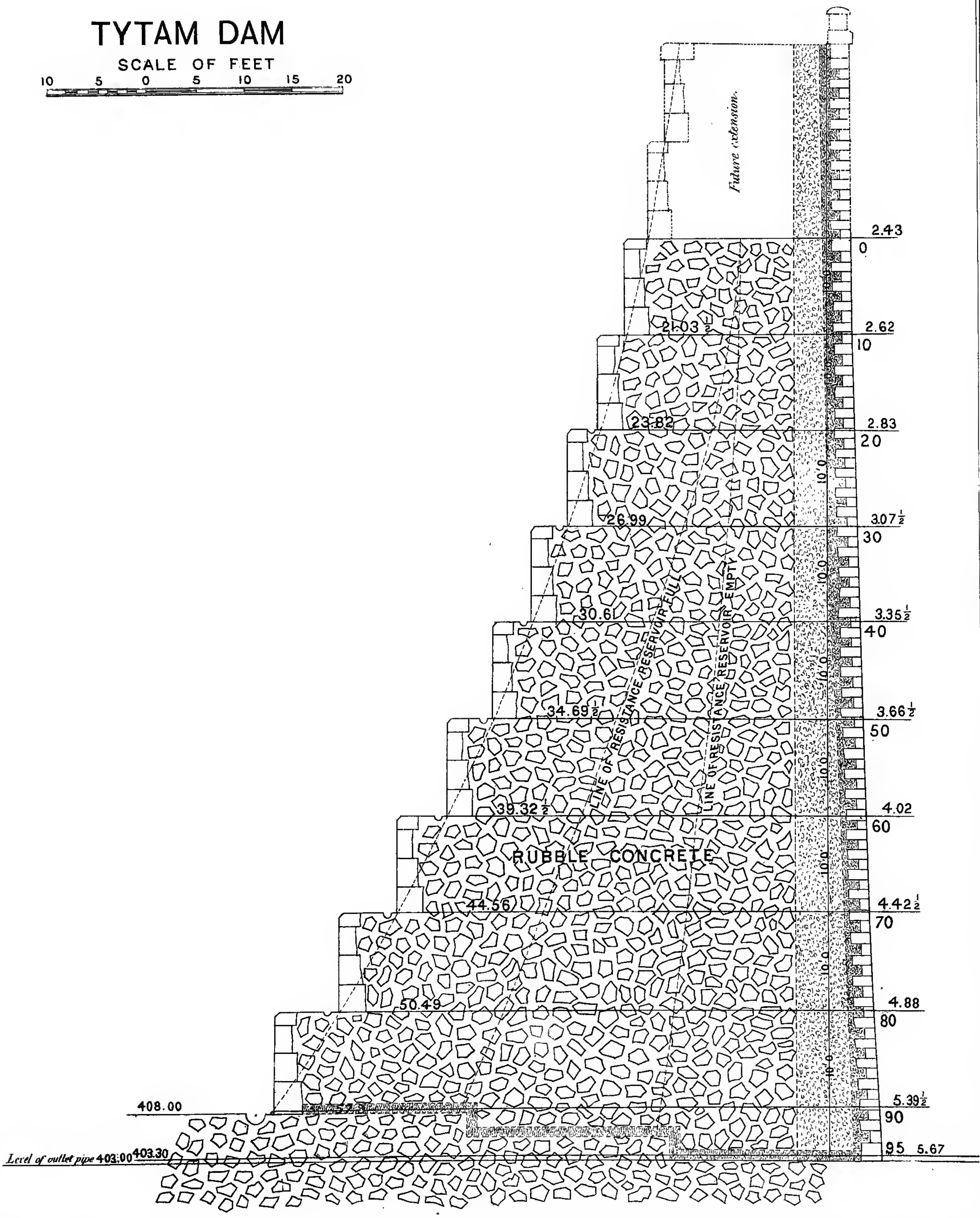
SEE PAGE 81.

GEELONG DAM

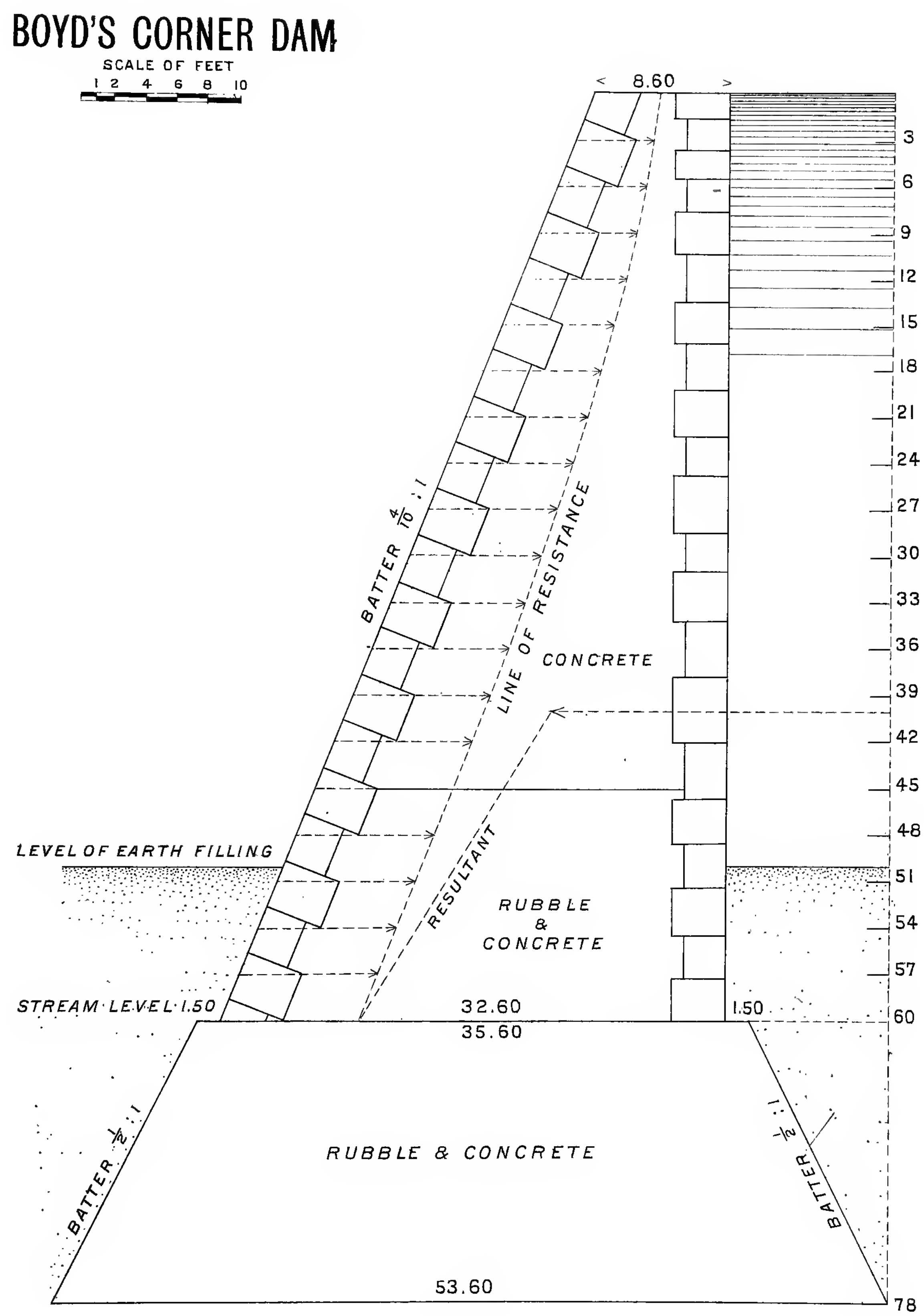


SEE PAGE 82.

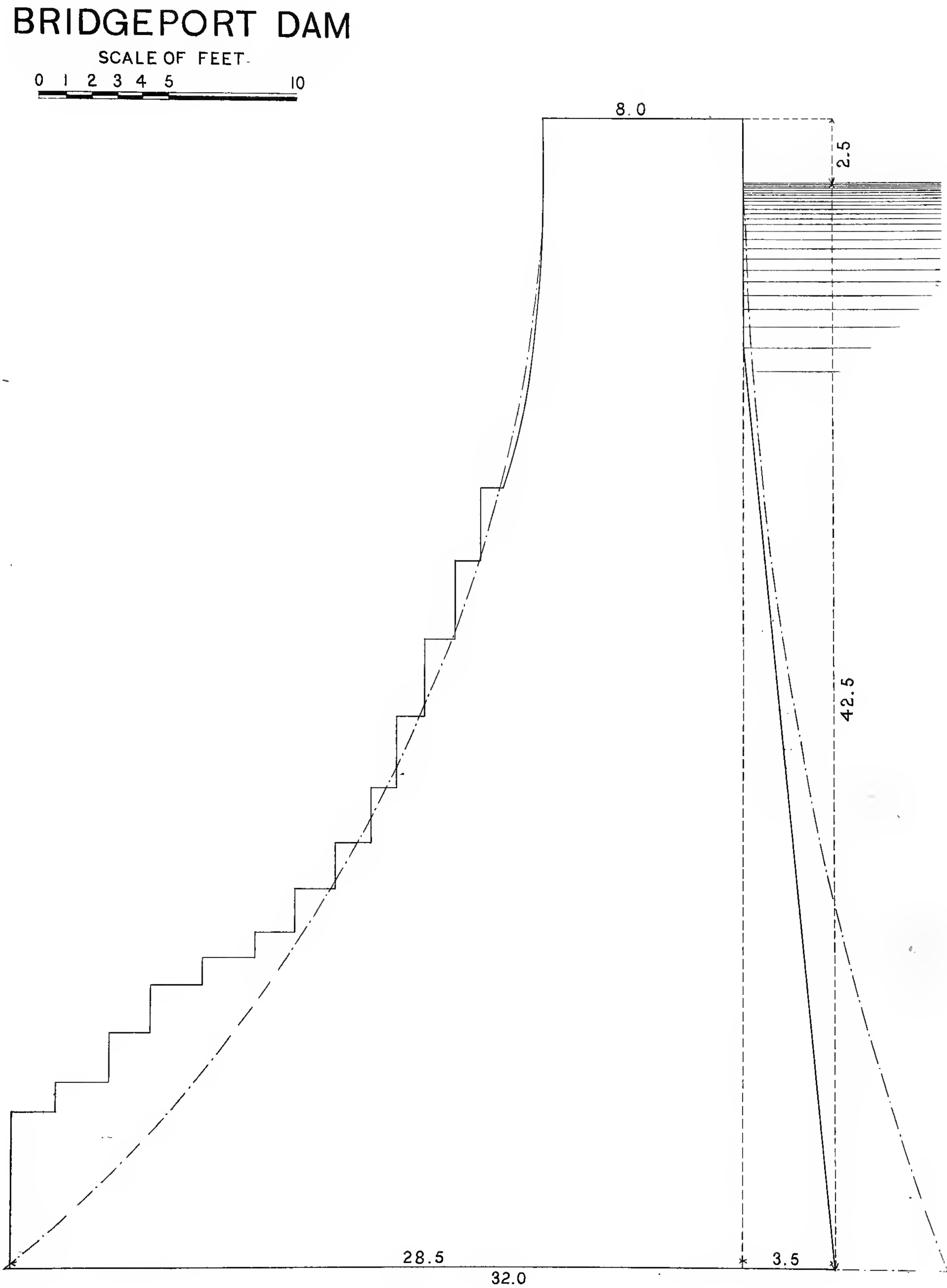
TYTAM DAM



SEE PAGE 84

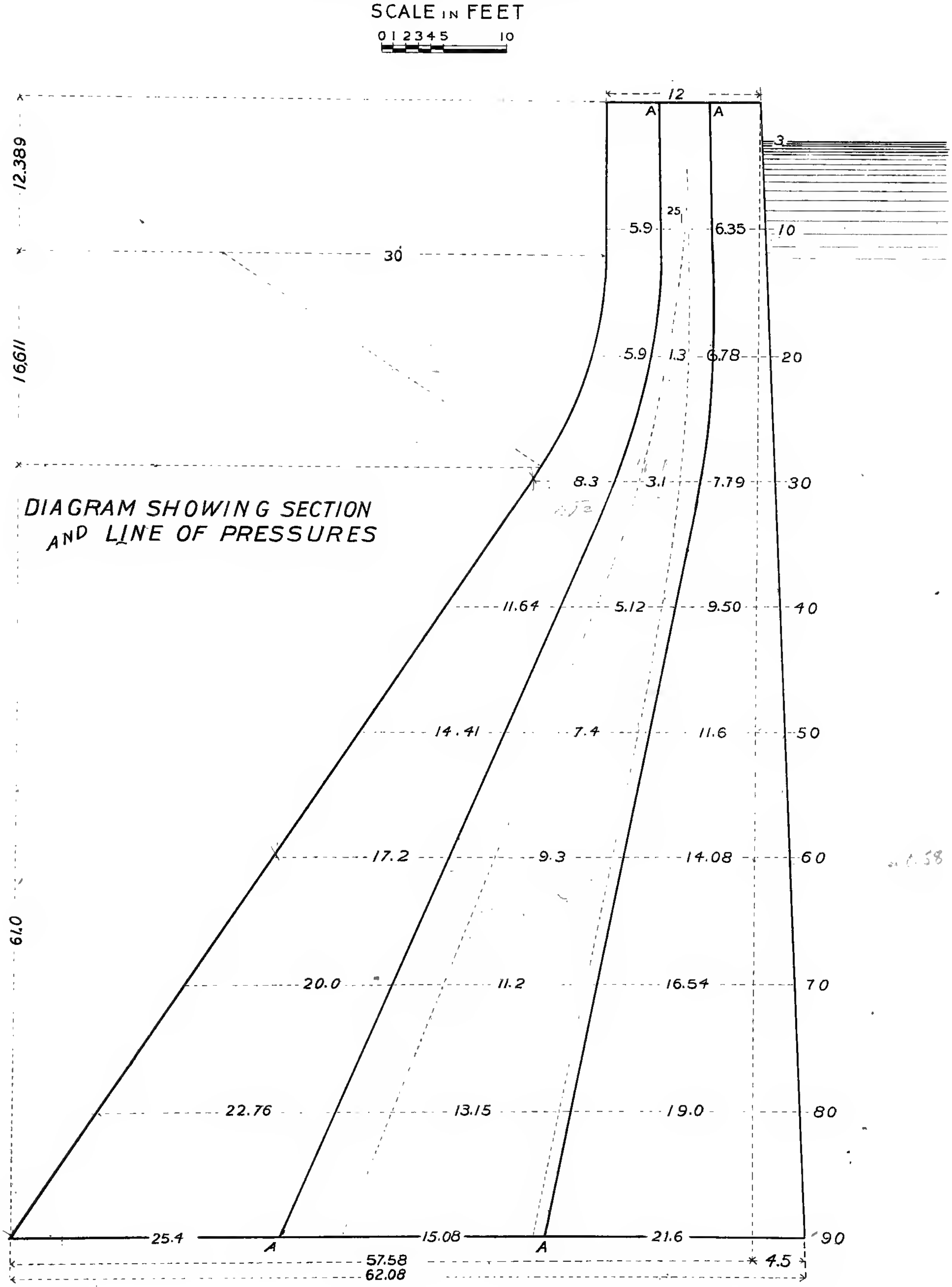

BOYD'S CORNER DAM
SCALE OF FEET
1 2 4 6 8 10
8.60
BATTER 4/10 : 1
LINE OF RESISTANCE
CONCRETE
RESULTANT
LEVEL OF EARTH FILLING
RUBBLE & CONCRETE
STREAM LEVEL 1.50
32.60
35.60
1.50
RUBBLE & CONCRETE
BATTER 1/2 : 1
BATTER 1/2 : 1
53.60
3
6
9
12
15
18
21
24
27
30
33
36
39
42
45
48
51
54
57
60
78

SEE PAGE 85



SEE PAGE 85

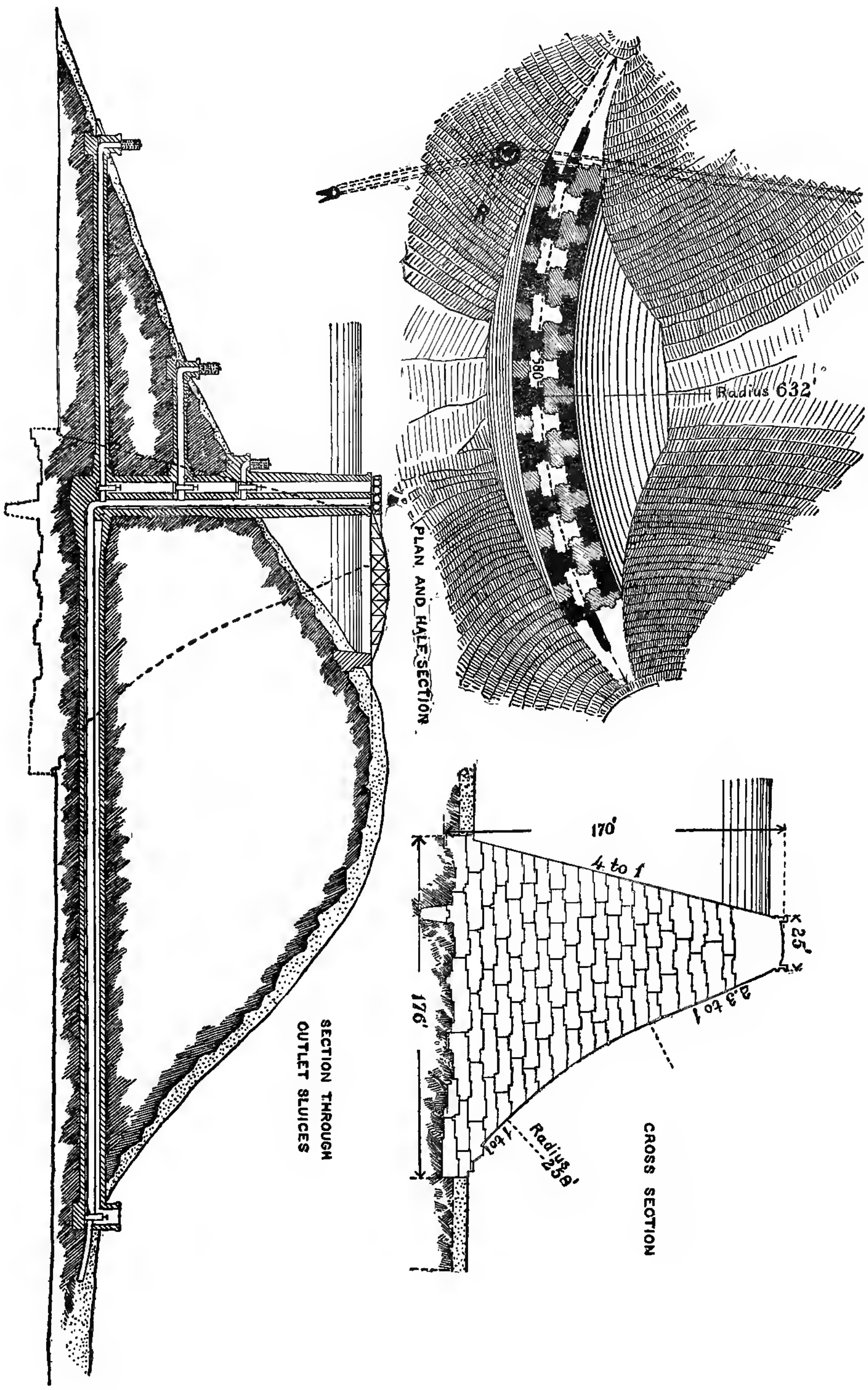
WIGWAM DAM



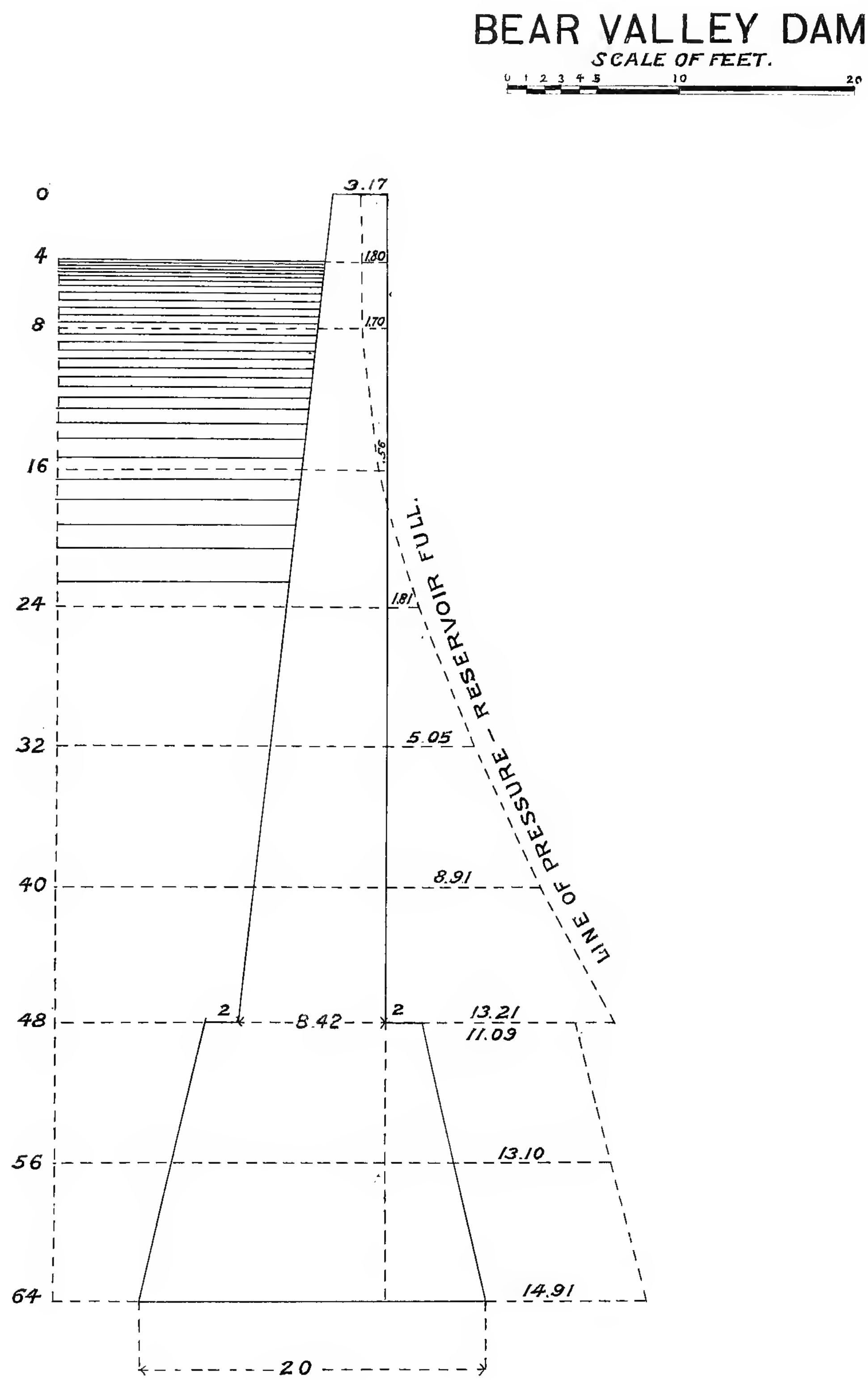
A—A. LINES DEFINING THE MIDDLE THIRD
WATER TAKEN AT 62.5 LBS. PER CU.F.T.
MASONRY " " 150 " " " "

SEE PAGE 86.

SAN MATEO DAM.

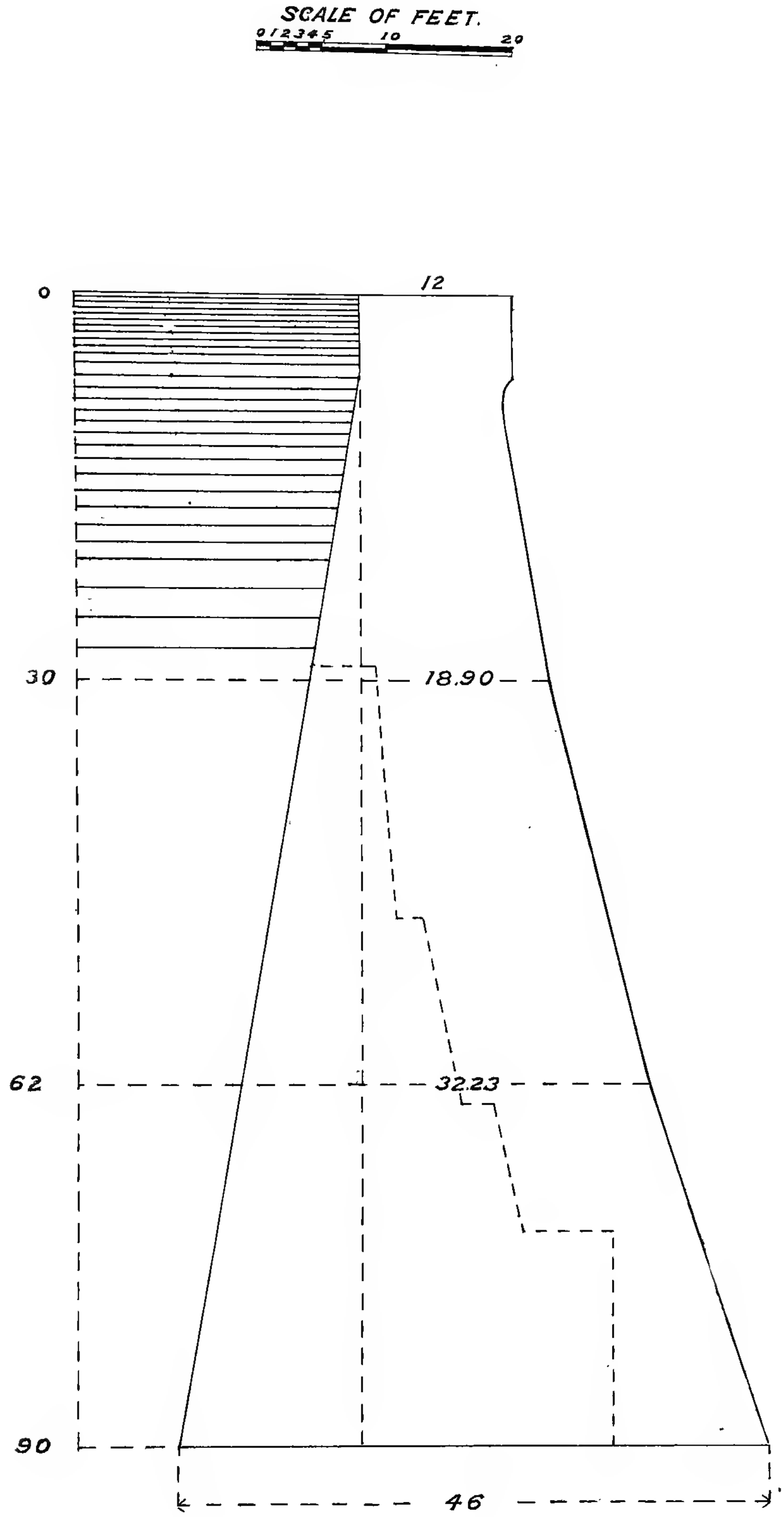


SEE PAGE 87


BEAR VALLEY DAM
SCALE OF FEET.
0 1 2 3 4 5 10 20
3.17
1.80
1.70
.56
1.81
5.05
8.91
13.21
11.09
13.10
14.91
8.42
2
2
20
LINE OF PRESSURE – RESERVOIR FULL.
0
4
8
16
24
32
40
48
56
64

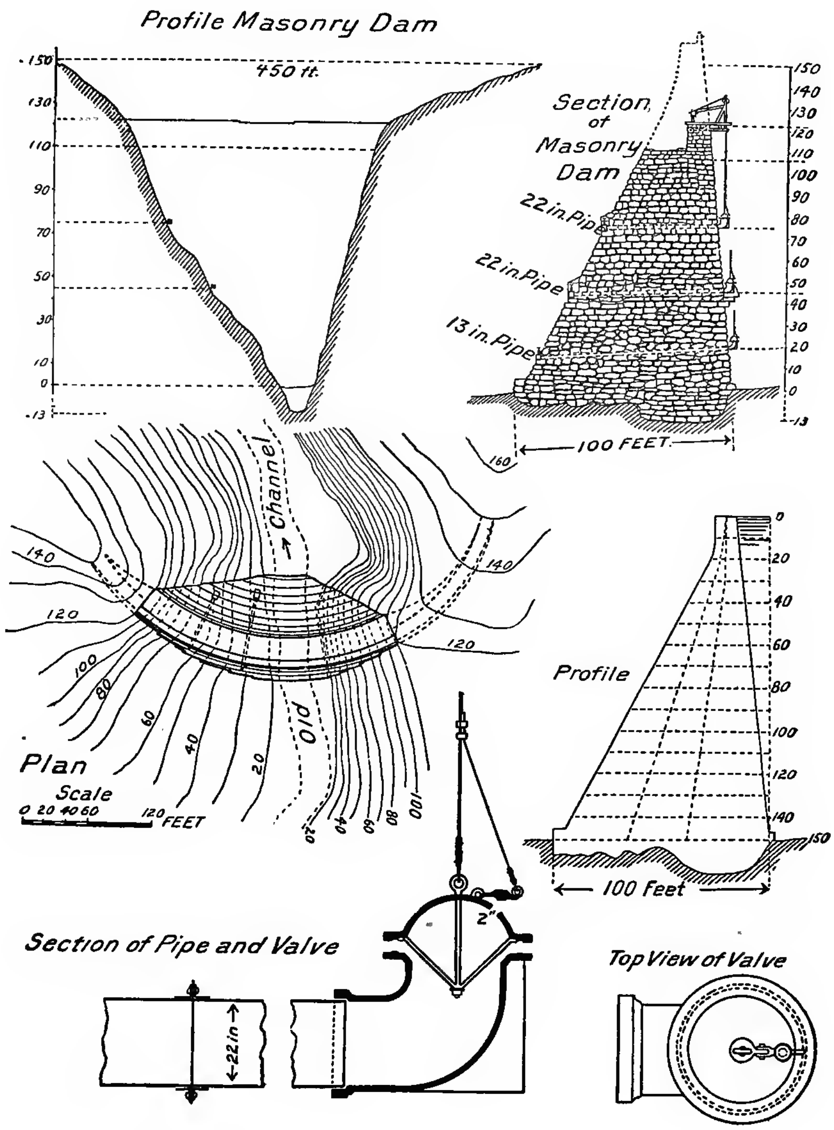
See Page 87

SWEET WATER DAM

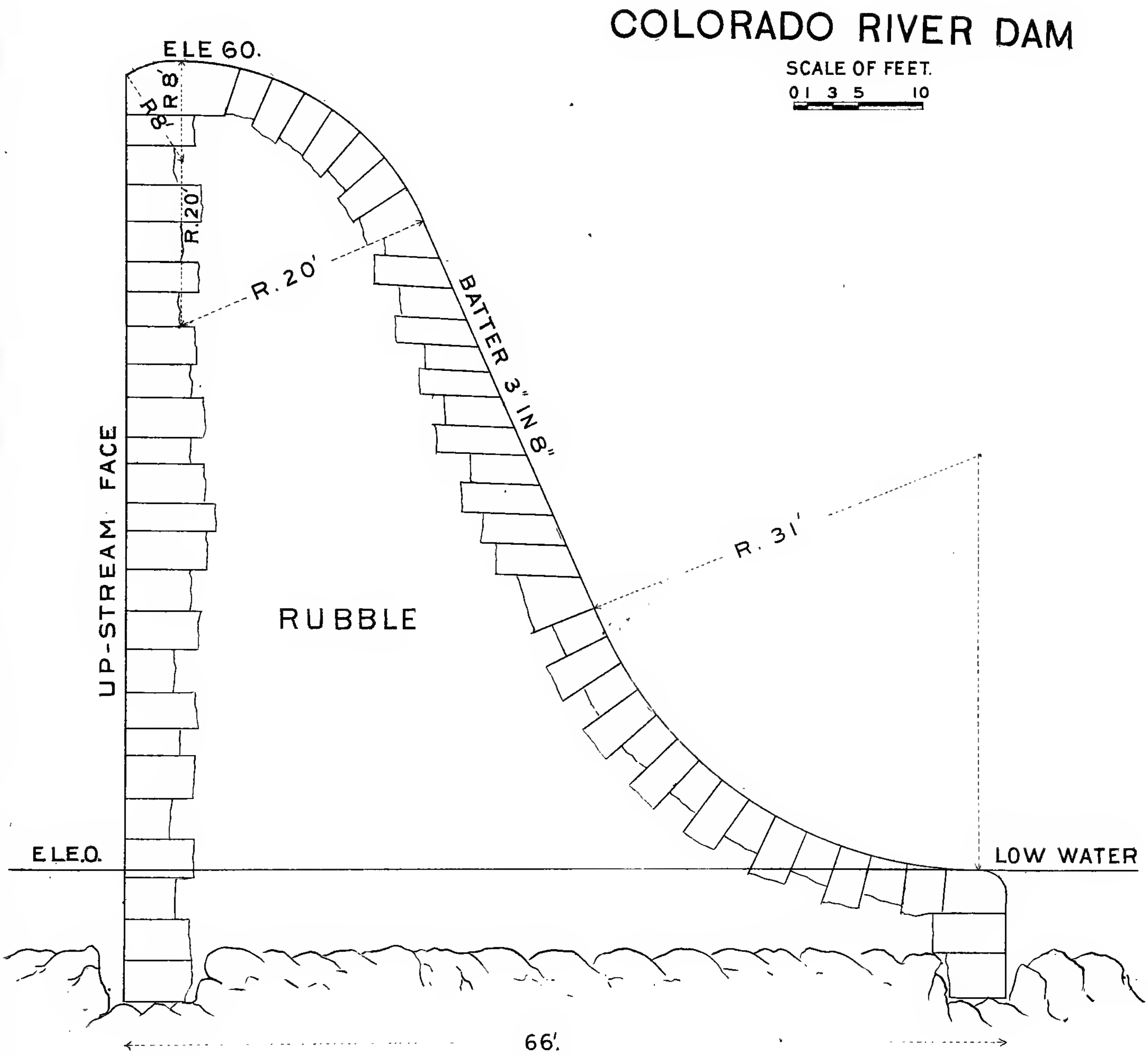


SEE PAGE 90.

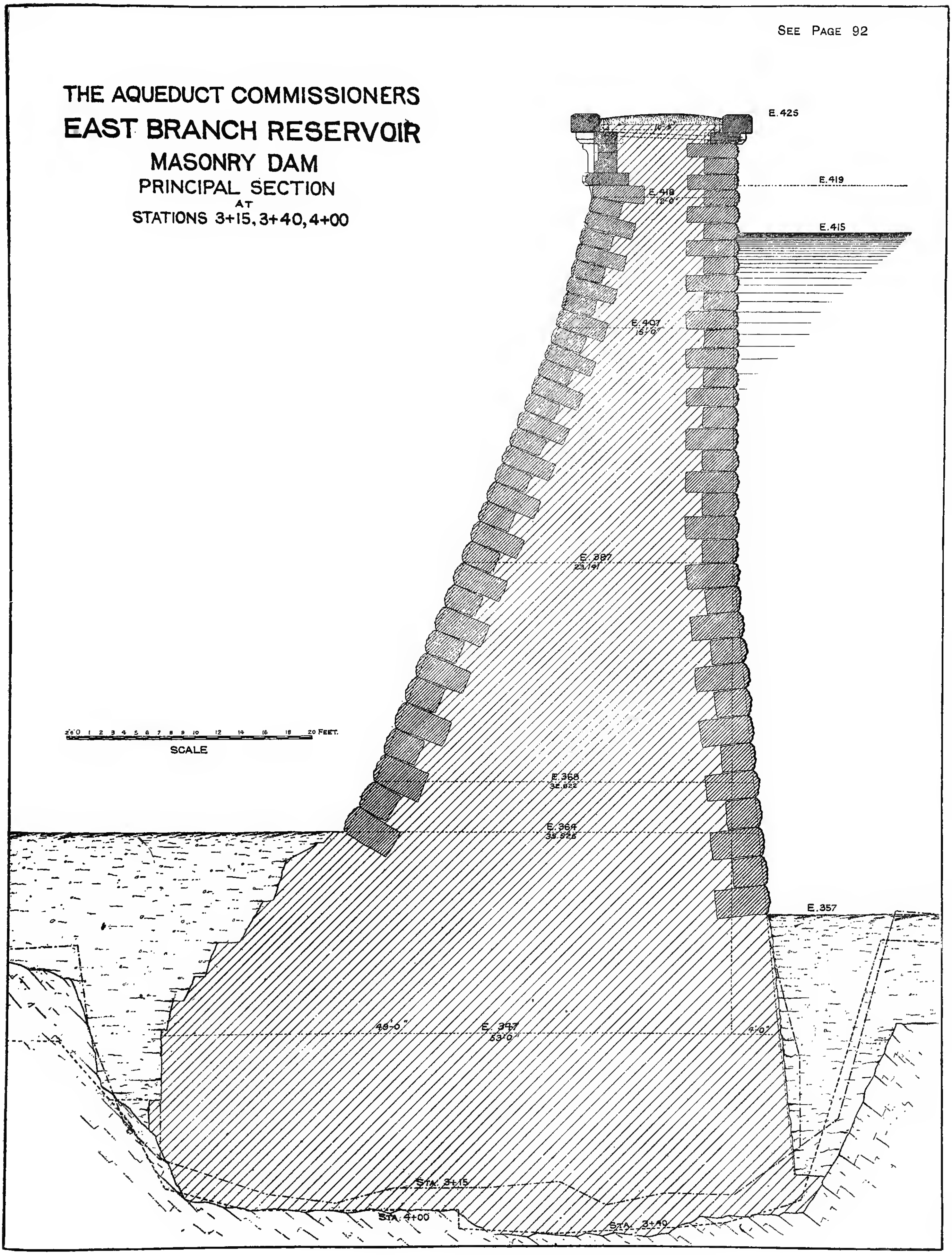
HEMMET DAM.



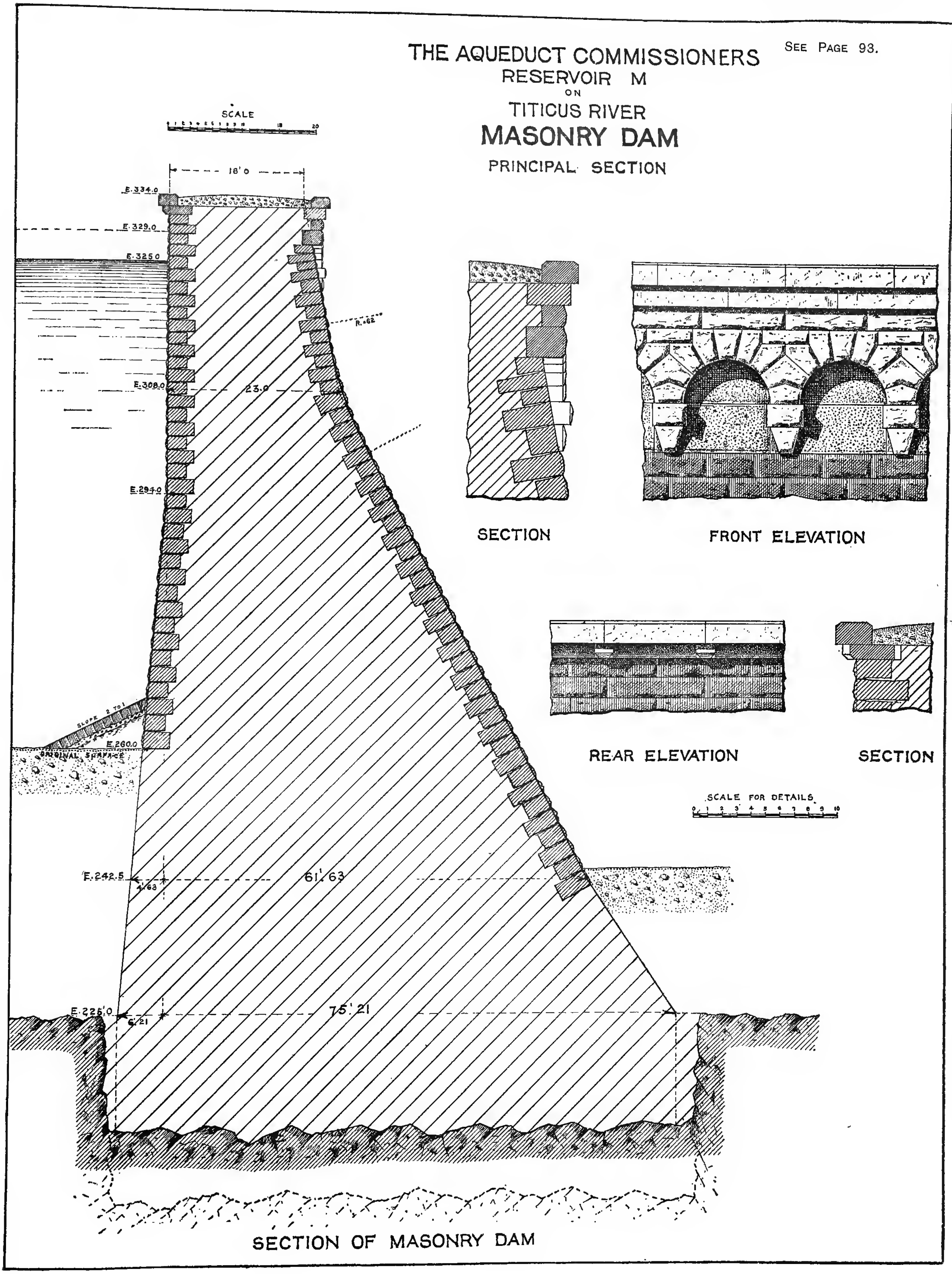
SEE PAGE 91


COLORADO RIVER DAM
SCALE OF FEET.
0 1 3 5 10
ELE 60.
R 8'
R 8'
R.20'
R. 20'
BATTER 3" IN 8"
R. 31'
UP-STREAM FACE
RUBBLE
ELE.O.
LOW WATER
66'.

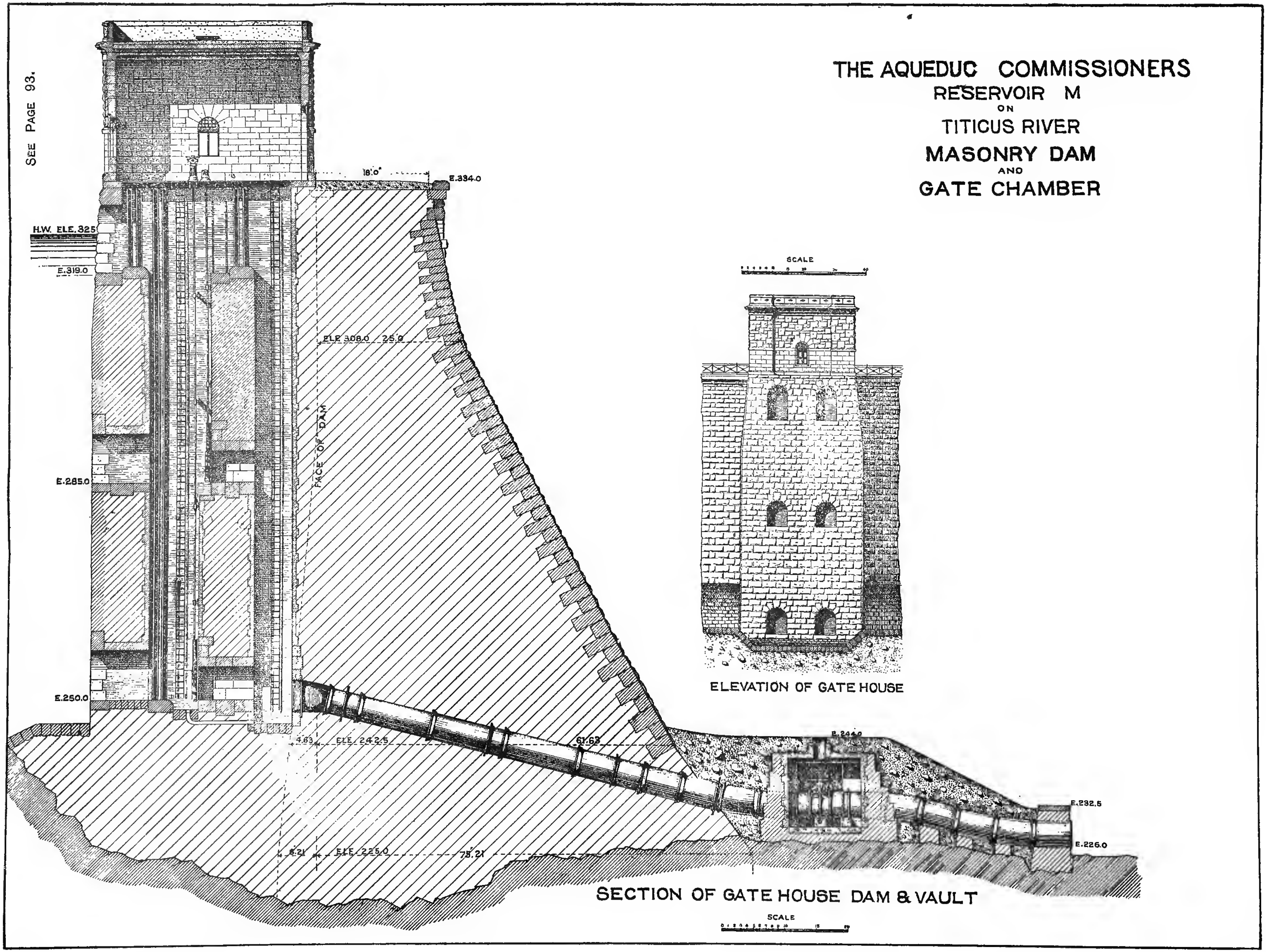
SEE PAGE 92



THE AQUEDUCT COMMISSIONERS
EAST BRANCH RESERVOIR
MASONRY DAM
PRINCIPAL SECTION
AT
STATIONS 3+15, 3+40, 4+00
E. 425
E. 419
E. 418
12'-0"
E. 415
E. 407
15'-0"
E. 387
23.191
E. 368
32.822
E. 364
35.526
E. 357
E. 347
53'-0"
49'-0"
4'-0"
STA. 3+15
STA. 4+00
STA. 3+40
20 FEET.
SCALE


THE AQUEDUCT COMMISSIONERS
RESERVOIR M
ON
TITICUS RIVER
MASONRY DAM
PRINCIPAL SECTION
SEE PAGE 93.
SCALE
18' 0
E.334.0
E.329.0
E.325.0
R.62
E.308.0
23.0
E.294.0
SLOPE 2 TO 1
E.260.0
ORIGINAL SURFACE
E.242.5
4.63
61.63
E.225.0
6.21
75.21
SECTION
FRONT ELEVATION
REAR ELEVATION
SECTION
SCALE FOR DETAILS
SECTION OF MASONRY DAM

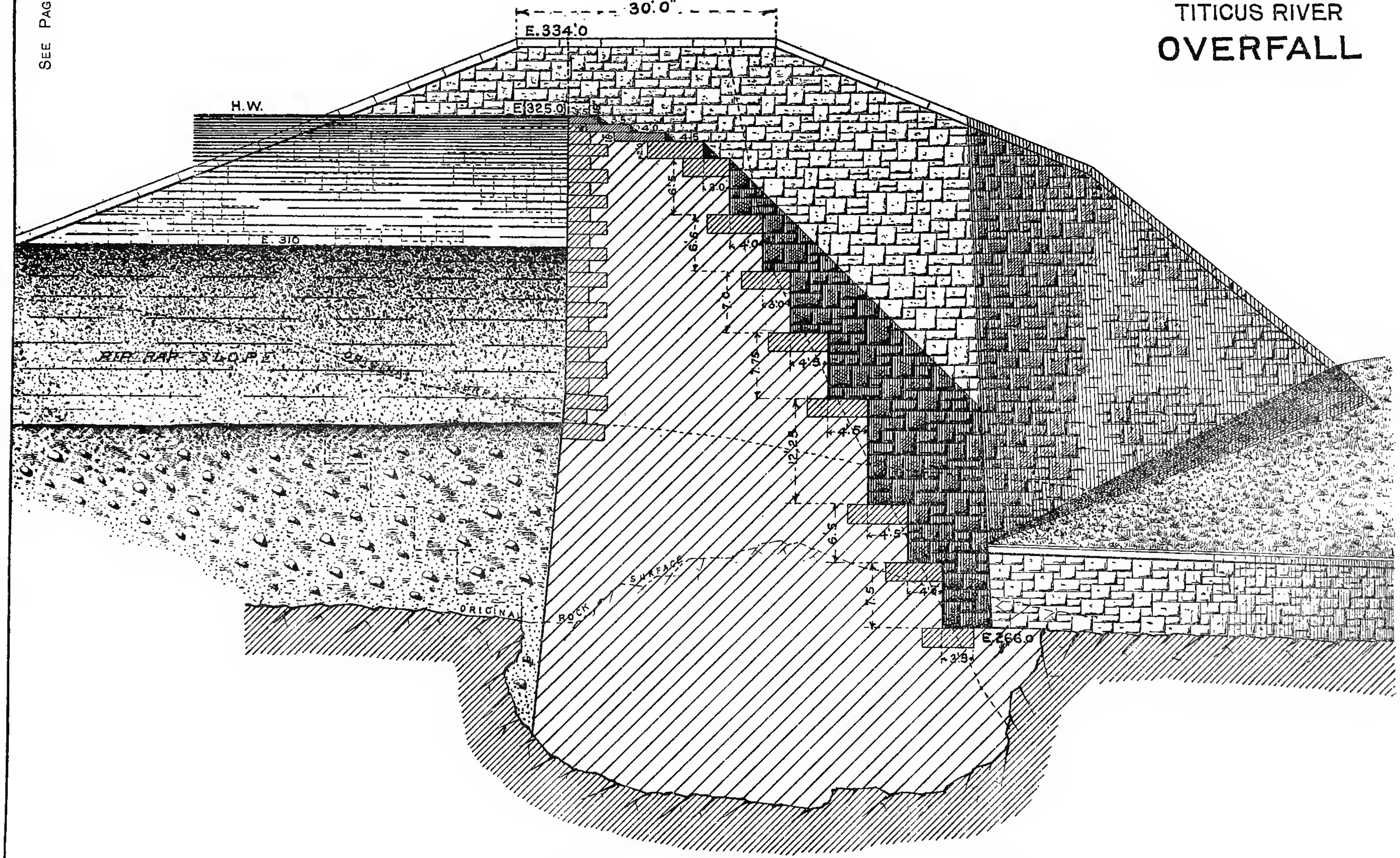
SEE PAGE 93.


THE AQUEDUC COMMISSIONERS
RESERVOIR M
ON
TITICUS RIVER
MASONRY DAM
AND
GATE CHAMBER
H.W. ELE. 325
E.319.0
E.285.0
E.250.0
E.334.0
18.0"
ELE 308.0 25.0
FACE OF DAM
4.63
ELE. 242.5
61.63
6.21
ELE. 225.0
75.21
E.244.0
E.232.5
E.226.0
SCALE
ELEVATION OF GATE HOUSE
SECTION OF GATE HOUSE DAM & VAULT
SCALE

SEE PAGE 94.

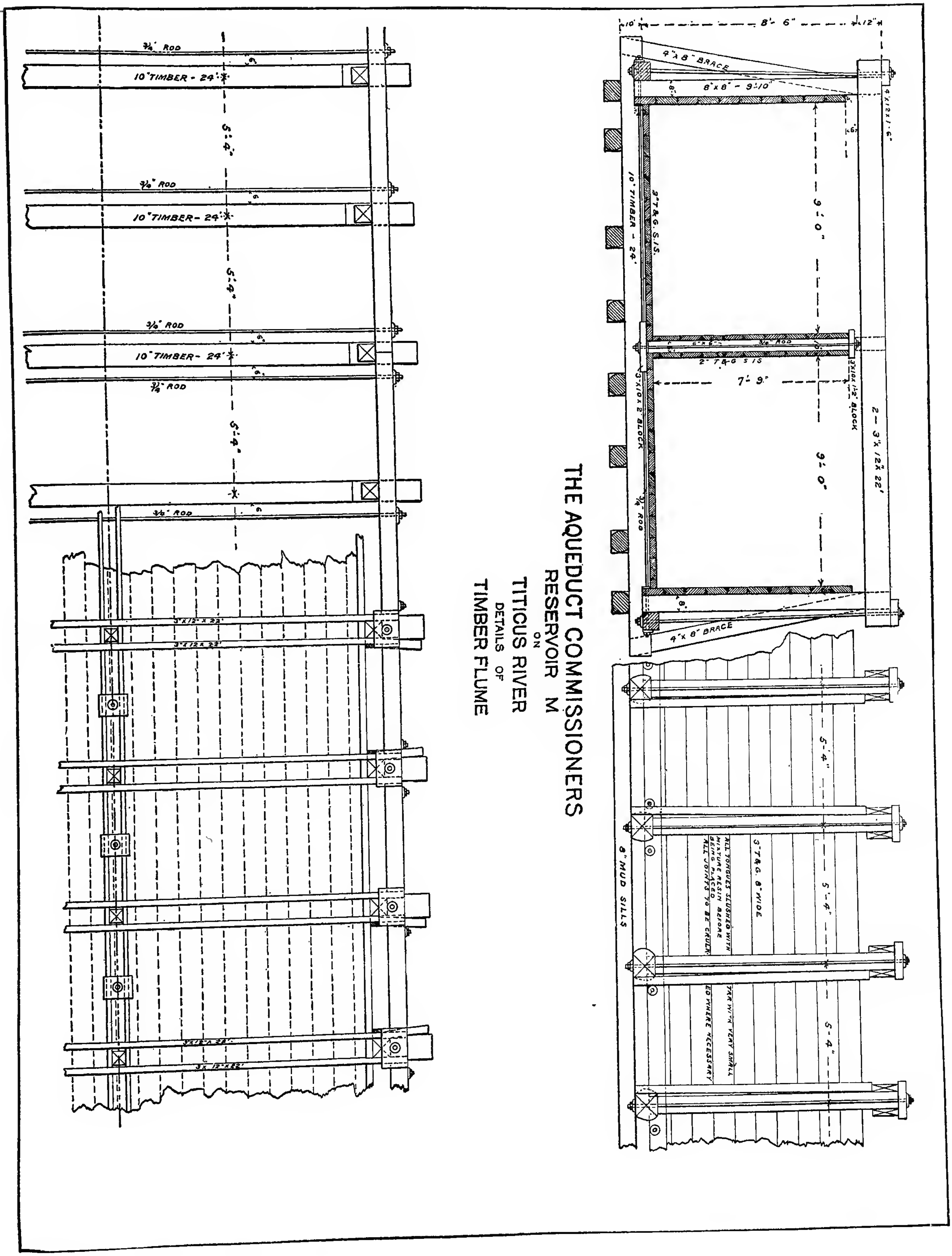
THE AQUEDUCT COMMISSIONERS
RESERVOIR M
ON
TITICUS RIVER

OVERFALL

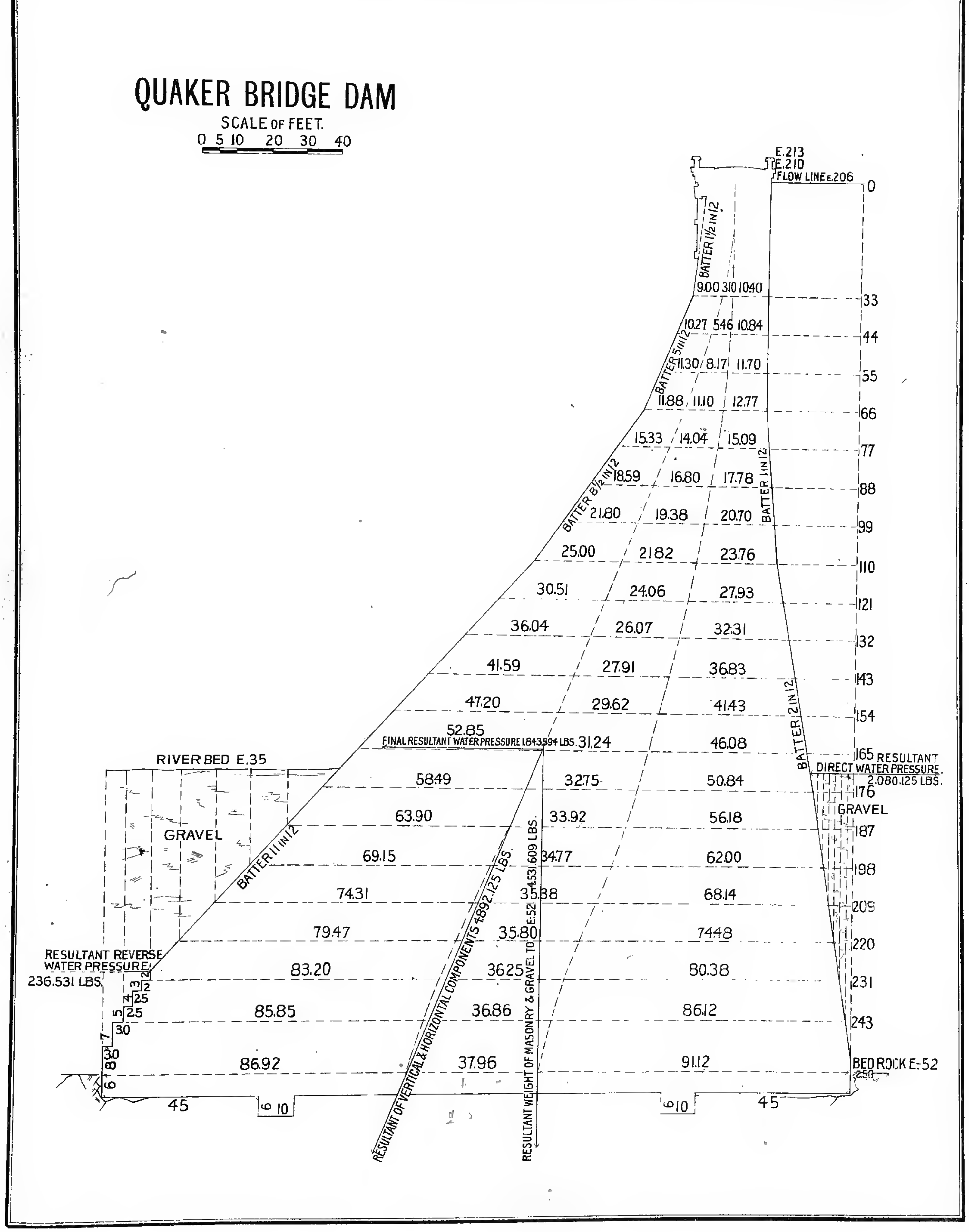


SECTION AT CENTER OF OVERFALL

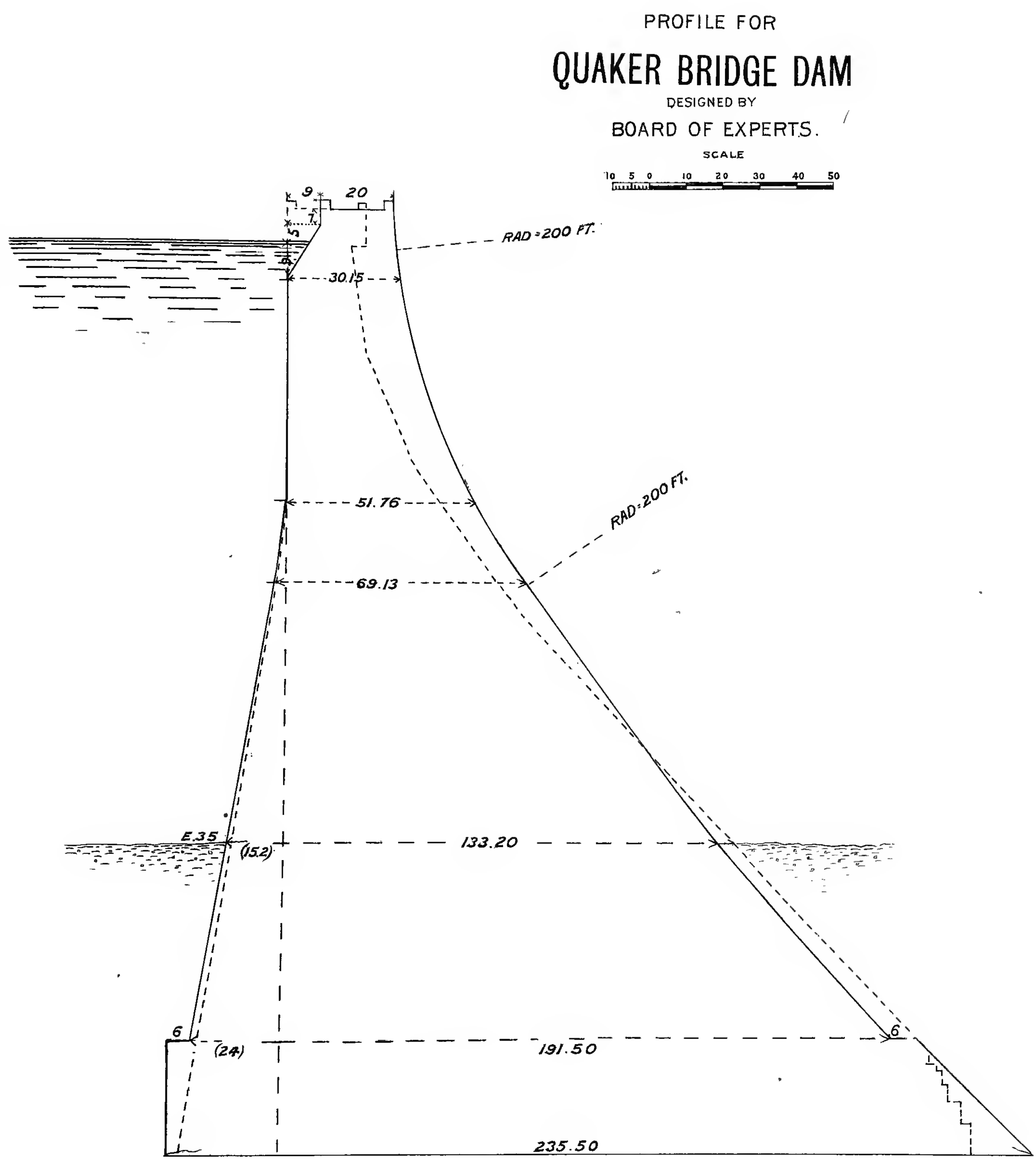
SCALE


THE AQUEDUCT COMMISSIONERS
RESERVOIR M
ON
TITICUS RIVER
DETAILS OF
TIMBER FLUME
10" TIMBER - 24'
5'-4"
3/4" ROD
3"x12"x22'
8'-6"
4"x8" BRACE
8"x8" - 9'-10"
9'-0"
7'-9"
3"T&G S1S
2" T&G S1S
10" TIMBER - 24'
2 - 3"x12"x22'
3"x10"x2' BLOCK
8" MUD SILLS
3" T&G. 8" WIDE
ALL TONGUES SLUSHED WITH TAR WITH VERY SMALL MIXTURE RESIN BEFORE BEING PLACED
ALL JOINTS TO BE CAULKED WHERE NECESSARY

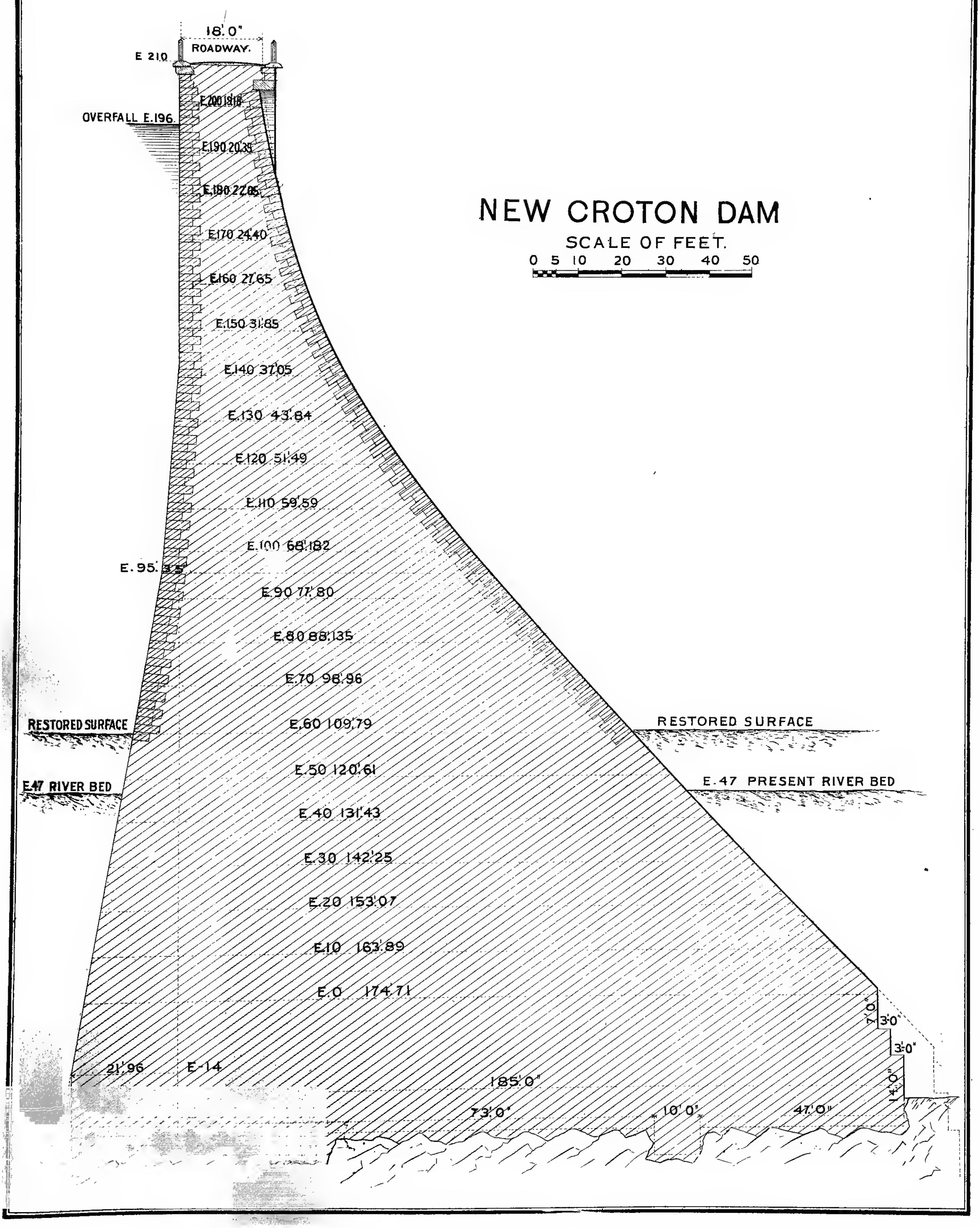
SEE PAGE 97


QUAKER BRIDGE DAM
SCALE OF FEET.
0 5 10 20 30 40
E.213
E.210
FLOW LINE E.206
BATTER 1½ IN 12
BATTER 5 IN 12
BATTER 8½ IN 12
BATTER 11 IN 12
BATTER 1 IN 12
BATTER 2 IN 12
9.00 3.10 10.40
10.27 5.46 10.84
11.30 8.17 11.70
11.88 11.10 12.77
15.33 14.04 15.09
18.59 16.80 17.78
21.80 19.38 20.70
25.00 21.82 23.76
30.51 24.06 27.93
36.04 26.07 32.31
41.59 27.91 36.83
47.20 29.62 41.43
52.85
FINAL RESULTANT WATER PRESSURE 1.843.594 LBS. 31.24 46.08
58.49 32.75 50.84
63.90 33.92 56.18
69.15 34.77 62.00
74.31 35.38 68.14
79.47 35.80 74.48
83.20 36.25 80.38
85.85 36.86 86.12
86.92 37.96 91.12
0
33
44
55
66
77
88
99
110
121
132
143
154
165
176
187
198
209
220
231
243
RESULTANT DIRECT WATER PRESSURE. 2.080.125 LBS.
GRAVEL
RIVER BED E.35
GRAVEL
RESULTANT REVERSE WATER PRESSURE. 236.531 LBS.
BED ROCK E.-52
2.50
45
6 10
6 10
45
RESULTANT OF VERTICAL & HORIZONTAL COMPONENTS 4.892.125 LBS.
RESULTANT WEIGHT OF MASONRY & GRAVEL TO E.52 4.531.609 LBS.

SEE PAGE 104.


PROFILE FOR
QUAKER BRIDGE DAM
DESIGNED BY
BOARD OF EXPERTS.
SCALE
10 5 0 10 20 30 40 50
9
20
7
5
14
RAD = 200 FT.
30.15
51.76
RAD = 200 FT.
69.13
E.35
(15.2)
133.20
6
(24)
191.50
6
235.50

SEE PAGE 105


NEW CROTON DAM
SCALE OF FEET.
0 5 10 20 30 40 50
18'.0"
ROADWAY.
E 210
OVERFALL E.196.
E.200 18.18
E.190 20.35
E.180 22.05
E.170 24.40
E.160 27.65
E.150 31.85
E.140 37.05
E.130 43.84
E.120 51.49
E.110 59.59
E.100 68.182
E. 95. 3.5"
E.90 77.80
E.80 88.135
E.70 98.96
RESTORED SURFACE
E.60 109.79
RESTORED SURFACE
E.47 RIVER BED
E.50 120.61
E.47 PRESENT RIVER BED
E.40 131.43
E.30 142.25
E.20 153.07
E.10 163.89
E.0 174.71
7'0"
3'0"
3'0"
14'0"
21'.96
E-14
185'0"
73'0"
10'0"
47'0"

SEE PAGE 106.

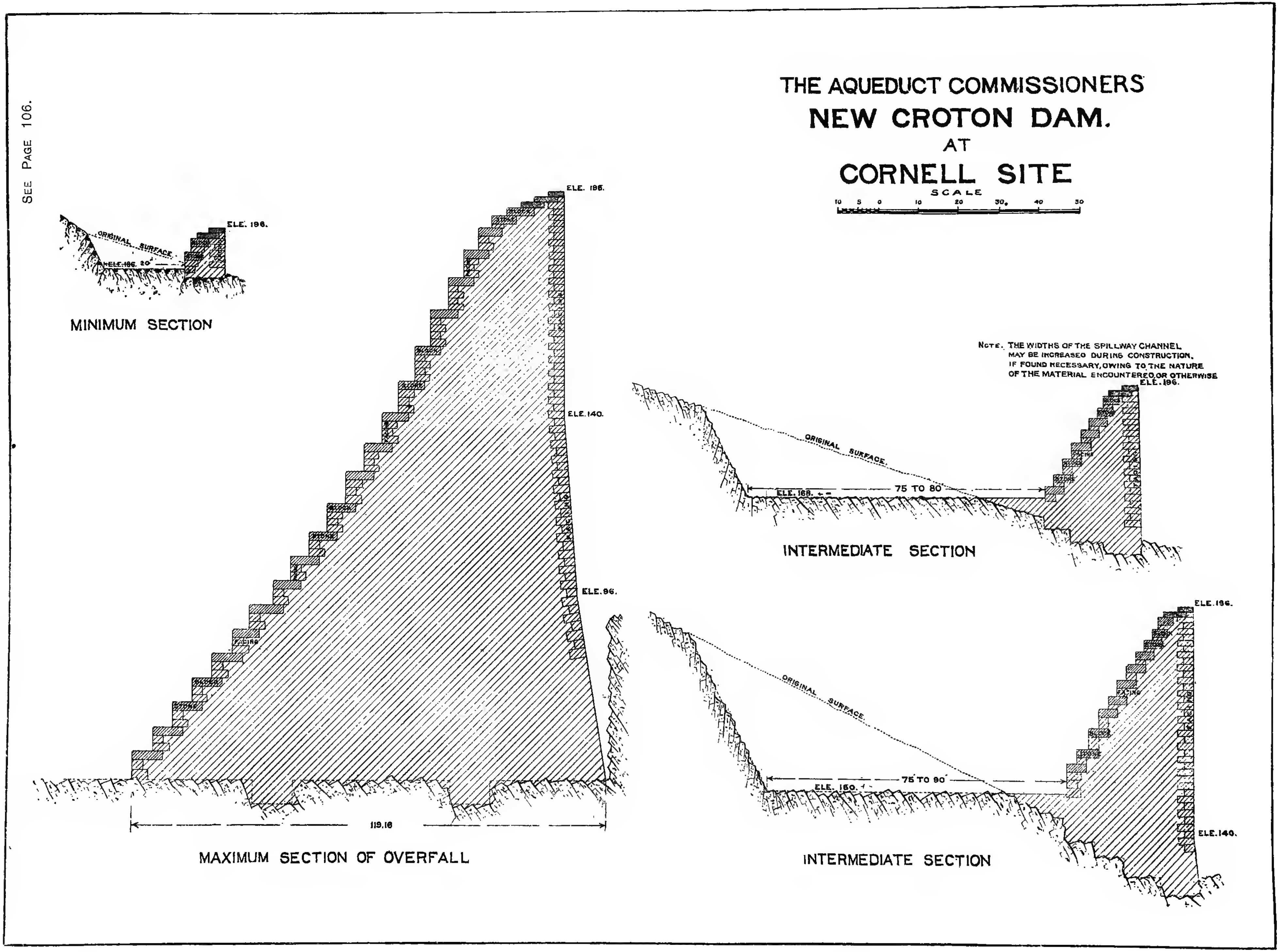
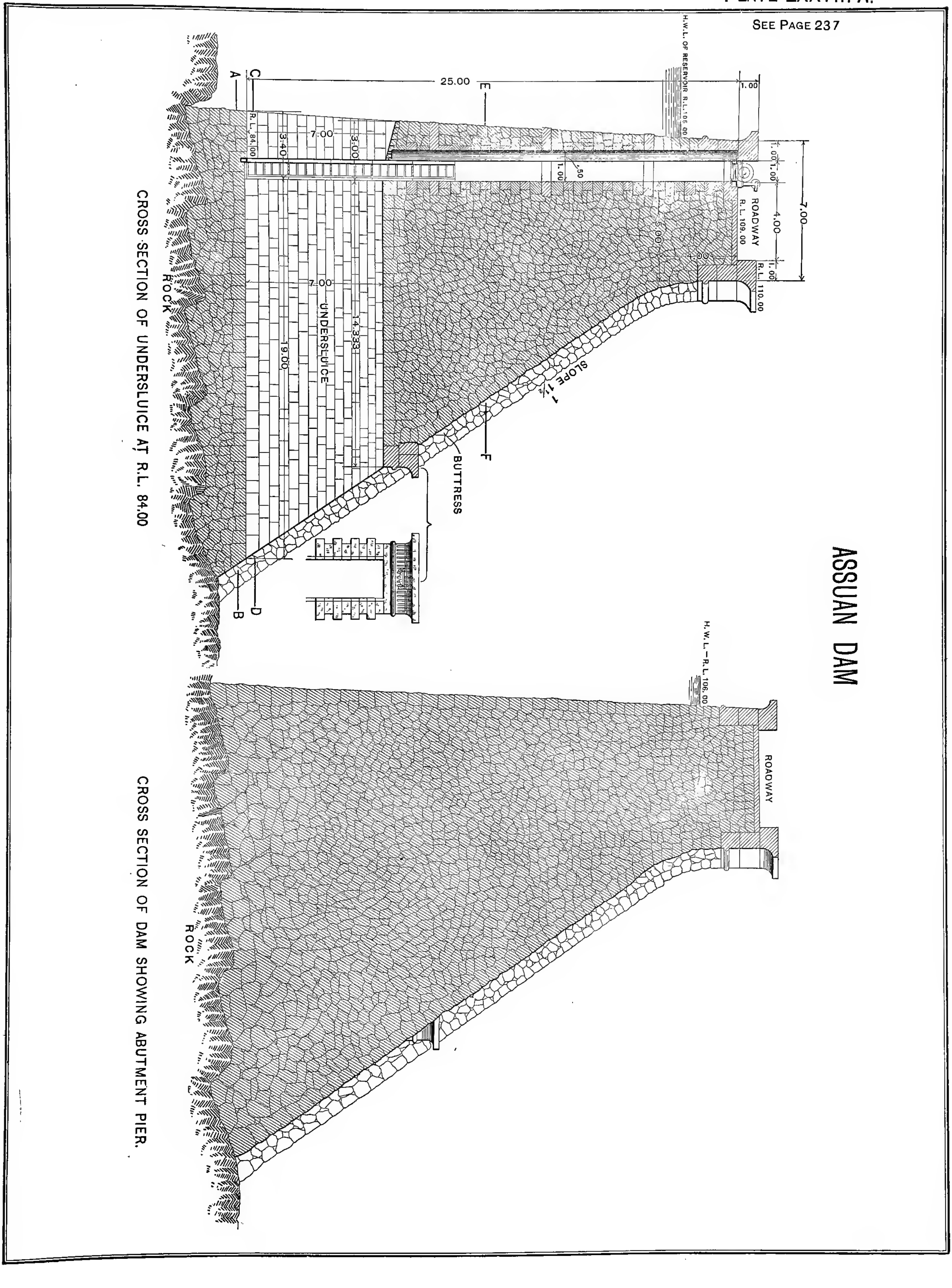

THE AQUEDUCT COMMISSIONERS
NEW CROTON DAM.
AT
CORNELL SITE
SCALE
10 5 0 10 20 30 40 50
ELE. 196.
ORIGINAL SURFACE
ELE. 186. 20
MINIMUM SECTION
ELE. 196.
ELE. 140.
ELE. 96.
119.16
MAXIMUM SECTION OF OVERFALL
NOTE. THE WIDTHS OF THE SPILLWAY CHANNEL MAY BE INCREASED DURING CONSTRUCTION, IF FOUND NECESSARY, OWING TO THE NATURE OF THE MATERIAL ENCOUNTERED OR OTHERWISE
ELE. 196.
ORIGINAL SURFACE
ELE. 168.
75 TO 80
INTERMEDIATE SECTION
ELE. 196.
ORIGINAL SURFACE
ELE. 160.
75 TO 90
ELE. 140.
INTERMEDIATE SECTION

PLATE LXXVIII A.

SEE PAGE 237



ASSUAN DAM

CROSS SECTION OF UNDERSLUICE AT R.L. 84.00

CROSS SECTION OF DAM SHOWING ABUTMENT PIER.

SEE PAGE 123.

THE AQUEDUCT COMMISSION

EARTH DAMS

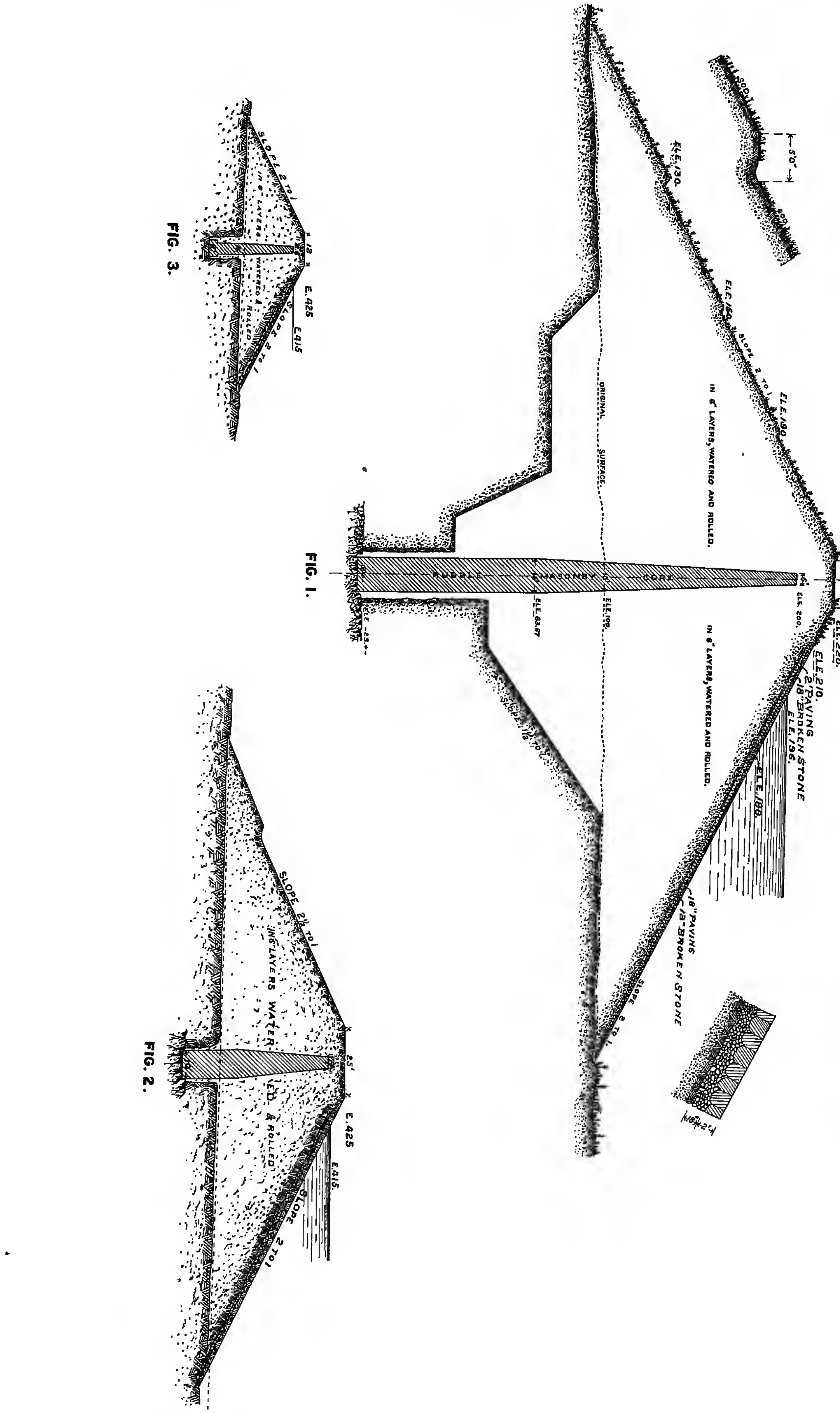


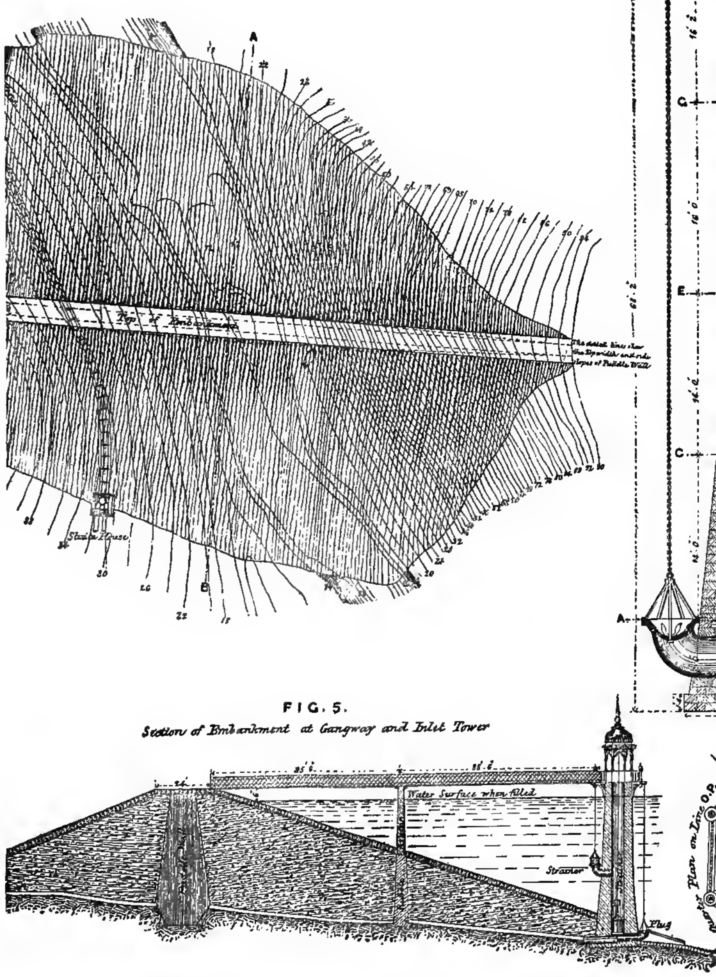
FIG. 1.

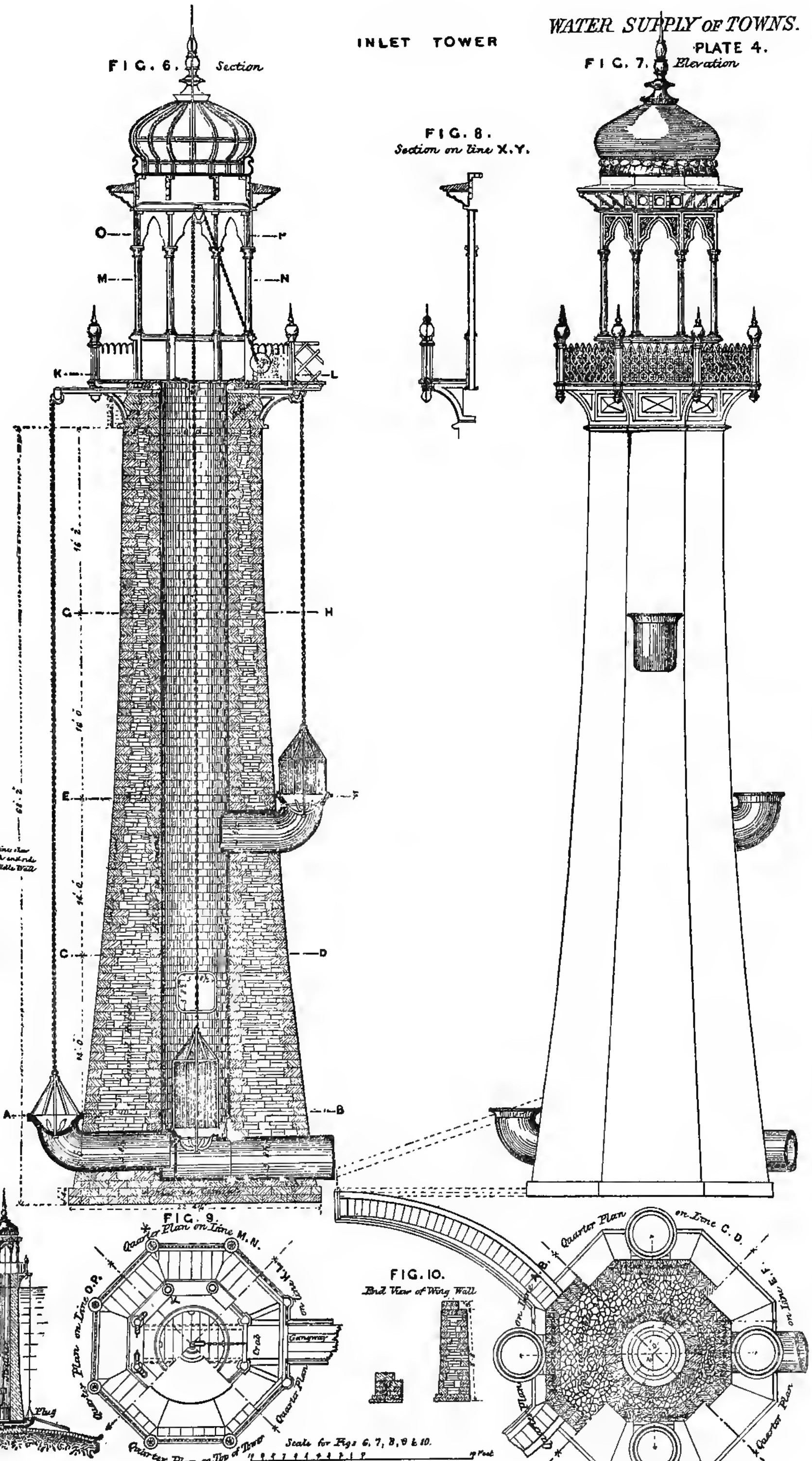
FIG. 2.

FIG. 3.

SEE PAGE 123.

BOMBAY WATERWORKS.





SEE PAGE 147.

HOLYOKE DAM.

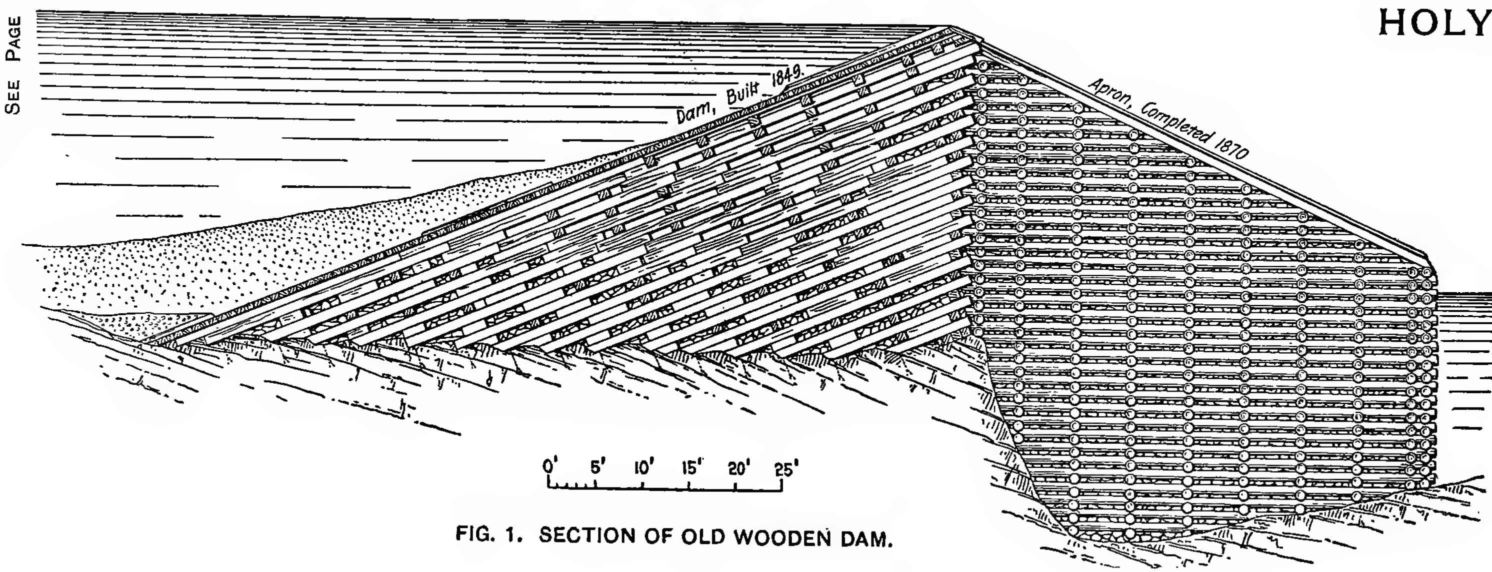


FIG. 1. SECTION OF OLD WOODEN DAM.

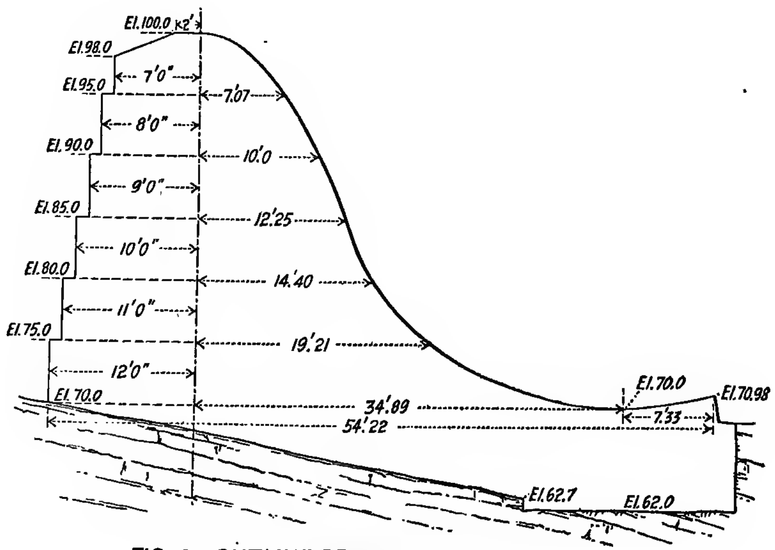


FIG. 2. OUTLINE SECTION OF DAM.

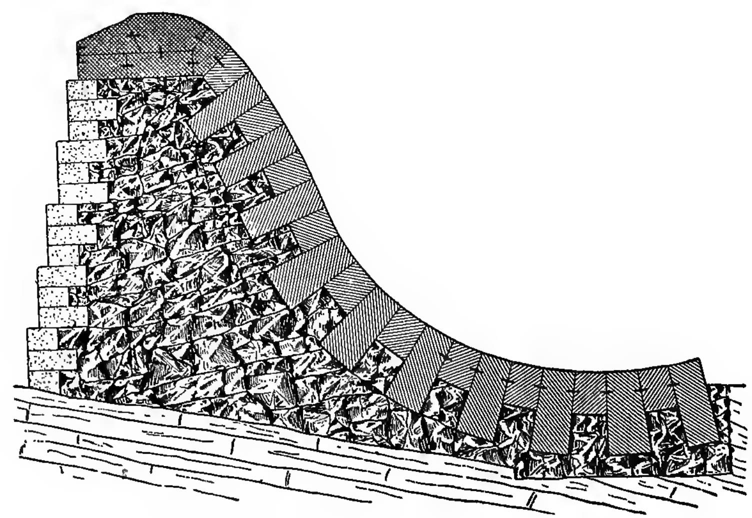


FIG. 3 CROSS SECTION OF DAM.